

57-F-239C

Hwy. # 17

WHITE RIVER EAST

BA 652

# TROW, SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS  
AND  
SOIL MECHANICS CONSULTATION

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Project: C 108/J117

September 13, 1957

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

57-F-239C

Attention: Mr. S. McGombie.

Foundation Investigation  
White River East Crossing  
T.C.H. No.17, Algoma District

Dear Sirs:

Attached hereto is our report on the recent foundation investigation carried out at the above described bridge site. The field work associated with this investigation was carried out during the period Aug. 2nd to Aug. 5th, 1957. A summary of the principal comments and recommendations contained in this report is as follows:

(1) The subsoil conditions at the abutment locations of the proposed structure are such that footings must be pile supported. It is recommended that bearing piles be driven to refusal in the dense stratum of sand and gravel intersected at an average depth of 30 feet below existing ground surface.

(2) Evidence of river bank undercutting at the north abutment location shows that moderate erosion can be expected in the immediate vicinity of the bridge site. It is recommended that rip rap be placed at the toe of the abutment breast walls and that erosion protection should be carried to elevation 1296 ft. on the approach embankment slopes. High water level is noted as 1294.4.

(3) An estimate of the horizontal earth pressures against the backwall of the abutment has been made assuming a sand and gravel backfill material. Thrusts of the order of 39 kips per abutment are indicated and it is considered that a total of 26 vertical piles are sufficient to accommodate this thrust at each abutment. The number could be reduced through the use of batter piles.

(4) Embankment foundation preparation should include complete removal of the surface mantel of organic topsoil and dry muskeg vegetation existing at this crossing site. The subsoil types are essentially granular and free draining and no problem in connection with embankment stability is anticipated.

We are pleased to have been of service to you on this occasion and trust that you will find the information contained herein sufficient for you to proceed with the design of the bridge abutments at this site. Should any questions arise out of the reading of this report we will be pleased to discuss them with you.

Yours very truly,

*L. G. Soderman*

Lawrence G. Soderman (P.Eng.)

LGS/lt  
Encls.

DEPARTMENT OF HIGHWAYS OF ONTARIO  
280 Davenport Road,  
Toronto.

FOUNDATION INVESTIGATION  
WHITE RIVER, EAST CROSSING  
T.C.H.#17, ALGOMA DISTRICT  
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ENCLOSURES

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FOUNDATION INVESTIGATION  
WHITE RIVER EAST CROSSING  
T.C.H. #17, ALGOMA DISTRICT

A boring program consisting of five boreholes and six dynamic cone penetration profiles has been completed at the above noted site. The description, extent and disposition of soil strata encountered are reported herein. Comments and recommendations regarding the type of footing support required at this bridge site are included for your consideration.

Location and Description of Site

The bridge site investigated is located on the Trans Canada Highway approximately 3 miles east of Obrian, Ontario, in the Algoma District. The structure is to span the White River which is crossed by Highway No. 17 at road centre line, chainage 616+20. This crossing has been designated the White River East Crossing.

The structure is to span the river at a point that corresponds to the centre of a bend in the stream channel. To the north of the bridge location the direction of flow is approximately west; to the south of the bridge site the direction of flow is approximately south-west. The velocity of flow is described as medium.

The change in direction of stream flow at the crossing site has resulted in slight undercutting of the river banks at the north abutment location. Local erosion of a lesser degree was also observed on the south bank of the river. The valley through which the White River flows is fairly broad and low lying at the crossing site. Spring flood waters overflow the river banks and flood level is recorded as elev. 1294.4 which is approximately 1 foot above the present bank elevation.

Extent of Field Work

In order to transport the drilling equipment to the bridge site it was necessary to repair the winter tote road from Obrian to the river crossing. This was done with the assistance of the road contractor's forces and a period of five working days was required. Drilling was started on borehole number 4 on Aug. 2nd, a.m.

The boreholes were carried out using a standard diamond drilling rig equipped to take soil samples. The holes were advanced using conventional wash boring procedure whereby a 3-inch diameter casing is alternately driven and washed out to the depths required. Samples were obtained at depth intervals of five feet or where a change in soil type was observed during drilling operations.

In the granular strata existing at this site, samples were taken using a 2-inch diameter split spoon. This was driven dynamically into the stratum being sampled using an energy of 350 ft.lb./blow. The above sample

dimensions and driving energy conform to the requirements of the empirical Standard Penetration Test used extensively in America and the United Kingdom to evaluate the bearing capacity of fine grained granular soils. This test is carried out in an open borehole and the results obtained are liable to serious error under the following two conditions:--(a) when sampling in water bearing sand strata, and (b) when washing procedure has been improperly carried out and results in disturbance of the stratum being sampled. Because of these limitations, dynamic cone penetration profiles were established adjacent to each borehole to supplement the Standard Penetration Test Results. These profiles are established by fixing a 2-inch diameter cone to the end of a Standard A drilling rod and then driving the cone and rods from ground surface to refusal depth. The driving energy is equivalent to that used in the S.P. Test and resistance to cone advance is recorded as number of blows required per foot of cone penetration.

The samples were examined and classified immediately upon recovery and then placed in moisture proof containers. On completion of the borings at this site the samples were transported by rail to the Toronto Laboratory where they will be stored for future reference if required.

Chaining and levelling required was carried out by the author. Elevations were referred to a D.H.O. Bench work located at Chainage 628+40, 35 ft. left of centre line, given as elevation 1313+97.

A total of 5 boreholes and 6 penetration cone profiles were carried out at this site. Field work was completed Aug. 5/1957.

#### Soil Types Encountered .

The stratigraphy of soil types occurring at the bridge site is shown on Drawing No.1. At the north abutment location a shallow surface mantel of organic topsoil and roots was found to overlie a stratum of loose brown silty fine sand. As evidenced by holes Nos. 1 and 2, the thickness of the layer averaged 10 feet, the relative density increased to medium dense at the lower horizon. At elevation 1281.9 and 1282.8 in holes 2 and 1 respectively, a layer of medium dense grey silt with very fine sand and small clay inclusions throughout was intersected. This material is essentially granular and non-plastic and is identical to material encountered at Wabikoba Bridge Site, described in report No. C108J113. The thickness of this strata averaged 20 feet in the vicinity of these two holes. Underlying the above described deposit a horizon of dense grey silty fine sand containing angular gravel and numerous boulders was encountered. Refusal to sampling and to dynamic cone penetration was obtained in this stratum. It is considered that refusal for bearing piles would be obtained at very small penetration into the material. The dense horizon was contacted at an average depth of approximately 30 feet below present ground surface. A layer thickness in excess of 12 feet was proven at hole No.1. .

At the south abutment location the surface mantel was found to consist of dry muskeg vegetation averaging  $3\frac{1}{2}$  feet in thickness. Underlying this surface deposit a stratum of brown silty fine to very fine sand containing organic and root fibres was intersected. A strong swampy odour was noted in the material from this layer. The thickness varied from 8 feet at holes Nos. 4

and 6 to 5.5 feet at hole No.5. In hole No. 4 a 4 foot layer of loose to medium dense very fine sand was found to underlie the organic deposit described above. In holes 5 and 6, the medium dense silt layer containing clay inclusions was contacted immediately below the organic silty sand stratum. The upper horizon of the silt deposit containing clay inclusions corresponded to elevations 1278.2, 1284.0 and 1282.4, in holes 4, 5 and 6 respectively. Elevation of the lower horizon of this stratum corresponding to locations as above, were 1256.7, 1257.5 and 1252.9. At these latter elevations the underlying dense horizon of silty sand with angular gravel and boulders was intersected. A thickness in excess of 9 feet was proven at hole No.4.

#### Consideration of Footing Support

Reference to the Standard Penetration Test results, and the currently accepted empirical evaluation of allowable spread footing loads based on these test values, shows that the subsoil at this site is not competent to support footing pressures in excess of 1 T/sq.ft. at economic excavation depths.

Bearing piles, either founded in the medium dense grey silt stratum containing clay inclusions or driven to refusal in the dense silty sand and gravel stratum, are considered the best means of supporting the abutments.

For piles driven to refusal in the underlying dense silty sand and gravel stratum the allowable pile load will be determined by the structural properties of the pile itself. Timber piles loaded to 15 to 20 tons per pile and steel monotube type piling loaded to 35 to 40 Tons per pile are typical values.

The safe allowable load on piles founded within the medium dense silt layer containing clay inclusions can be estimated from the following expression, published by Meyerhof<sup>(1)</sup>.

$$Q_s = \frac{4 N A_p}{F} + \frac{\bar{N} A_s}{KF} \quad \text{Tons /pile for } D/B \geq 10$$

where  $N$  = average Standard Penetration Test resistance near pile point

$\bar{N}$  = average Standard Penetration Test resistance within depth penetrated by pile.

$A_p$  = sectional area (Sq.ft.) of pile toe

$A_s$  = surface area of pile shaft

$K$  = constant - 100 for small displacement piles (e.g. H piles)  
- 50 for large displacement piles (e.g. timber piles)

$D$  = Depth of pile penetration in feet

$B$  = pile diameter in feet

$F$  = Safety Factor = 3.

SUPER IMPOSED DOCUMENT MAY  
APPEAR AS MULTI-FEED ON FILM.

(1) G.G. Meyerhof "Penetration Tests and Bearing Capacity of Cohesionless Soils". Proceedings, ASTM Vol. 82, No. 366, 1956.



The above expression contains two terms, the first represents the point resistance and the second represents the resistance due to shaft friction. In practice, the shaft friction only accounts for approximately 10% of the total capacity of a bearing pile and in the following pile capacity estimates this term has been neglected. Timber piles are assumed with a diameter of 10 inches at the tip. For piles of different dimensions the tabular value can be adjusted in proportion to the area ratio.

Location	File tip Elevation	Estimated Capacity Tons/pile
North Abutment Holes 1 and 2	1277.8 or below	8.5 T/pile
South Abutment Holes 4 and 5	1268.8 to 1263.	8.5 T/pile
South abutment Holes 4 and 5	1263 or below	14.5 T/pile

The above values incorporate a factor of safety of three and assume a  $D/B \geq 10$ . With a nominal spacing of  $2\frac{1}{2}$  to 3 pile diameters centre to centre of piles, the settlement of a pile group will not appreciably exceed that of an individual pile (i.e. 1" for above tabulation loads).

#### Horizontal Thrusts due to Abutment Back fill

In estimating the horizontal thrusts due to abutment backfill the active fill height has been taken as 10 feet. A granular free draining backfill material has been assumed having a unit weight of 110 p.c.f. and angles of shearing resistance and wall friction equal to  $\phi = 35^\circ$  and  $\delta = 20^\circ$  respectively. The above assumptions result in an active horizontal earth pressure coefficient of  $K_A = 0.23$ .

The total thrust per foot of backwall is given by the following expression:

$$P_H = K_A \frac{1}{2} \gamma H^2$$

where  $K_A$  = active earth pressure coefficient

$\gamma$  = unit weight of fill = 110 p.c.f.

H = 10 ft. = active fill height.

Substitution gives  $P_H \approx 1300 \text{ #/1.ft.}$

For a normal abutment width of 30 feet the total thrust is  $30 \times 13 = 39$  kips. If vertical piles are used a total of 26 piles would be required allowing a typical value of 1.5 kips per vertical pile.

If fewer piles than 26 are desirable then the designer can select a combination of vertical and batter piles in accordance with the abutment loading and desired footing sizes.

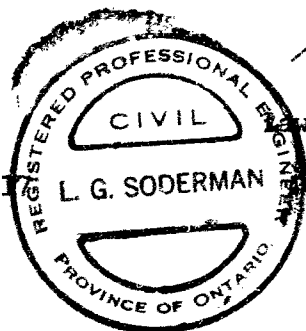
### Embankment Stability

Foundation preparation for the bridge approach embankments on both sides of the river should include stripping of the organic surface layers indicated on the profiles. This will involve a maximum stripping depth of 3.5 ft. The underlying granular materials are essentially free draining and settlements associated with embankment loading will for the most part be complete at the end of embankment construction. The subsoil types encountered in the borings are sufficiently competent to support the proposed embankment heights at this crossing site and no stability problems are anticipated.

### Comments and Recommendations

- (1) The subsoil conditions as evidenced by the borings and penetration tests carried out in the investigation show that abutment footings must be pile supported. It is recommended that bearing piles be driven to refusal in the underlying dense stratum encountered at an average depth of 30 feet below existing ground surface. An estimate of capacities for piles founded above this stratum is included in the body of this report. Cut off elevation for timber piles should be at or below elev. 1288.0 ft.
- (2) High water elevation is noted as 1294.4 which is slightly above present ground surface elevation at the abutment locations. Embankment slope should be protected to a minimum of 2 feet above this elevation in the immediate vicinity of the abutments. Slight under-cutting of the north river bank was noted and rip rap at the face of the north abutment is recommended.
- (3) An estimate of the horizontal thrusts resulting from abutment backfill has been made. A total thrust of 39 kips is indicated and it has been shown that this can be accommodated by 26 vertical piles. A free draining fine grained material should be specified as abutment backfill.
- (4) The organic surface deposits underlying the approach embankment locations should be stripped prior to placing of fill material. No problems in connection with embankment stability are anticipated.

LGS/lt  
September 13 1957  
Project: C 108 J 11



*L. G. Soderman*

Lawrence G. Soderman (P.Eng.)



North Abutment Location Looking  
Downstream

AUG 57



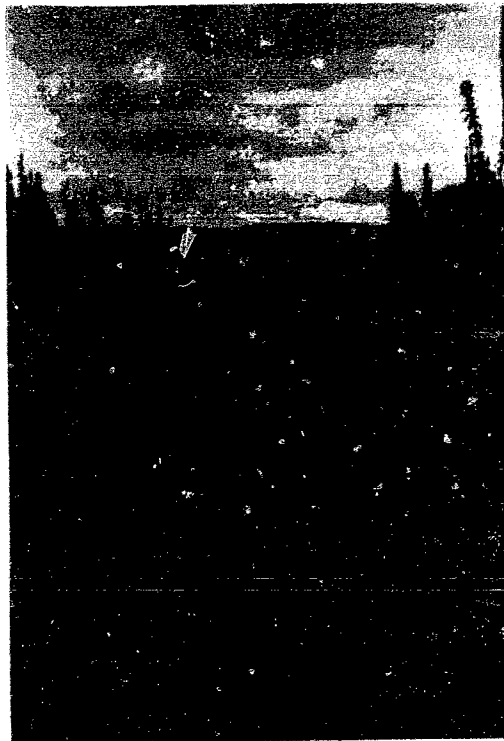
View showing Bridge Site, looking South  
from North Abutment Location

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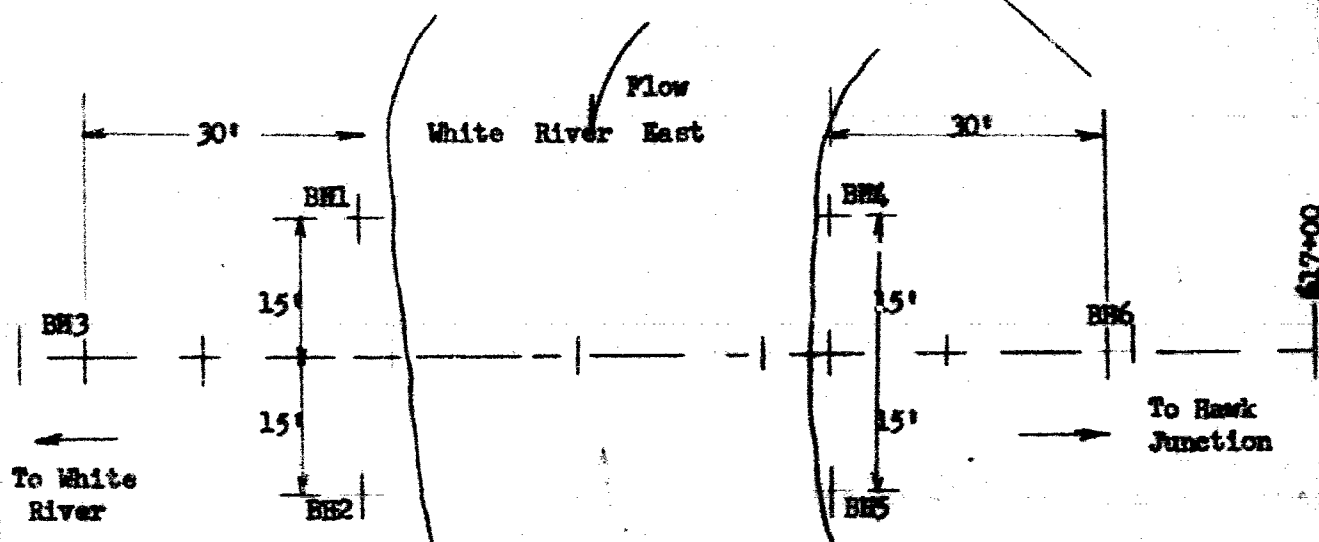
North Abutment Location Looking  
Downstream

AUG 57



View showing Bridge Site, looking South  
from North Abutment Location

SUPER IMPOSED DOCUMENT MAY  
APPEAR AS MULTI-FEED ON FILM.



BH 3	BH 2	H.W.L. 1294.4	BH 5	BH 6	1295
Topsoil & root fibres					
Loose brown silty fine sand		W.L. 1289.6 Aug. 4/51	Dry Muckey Loose brown organic silty very fine sand		1285
Loose to medium dense grey silty very fine sand					
		Medium dense grey silt with some very fine sand. Small dark grey clay inclusions throughout.			1275
					1265
1263.7		Dense stratum contact			
		Dense grey silty sand with angular gravel & boulders			1255
	1253.94				
			1248.5	1248.94	
+60	+80	+20	+40	+60	+80
612+00					612+00

PROJECT NO. 108 / J 117

# TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

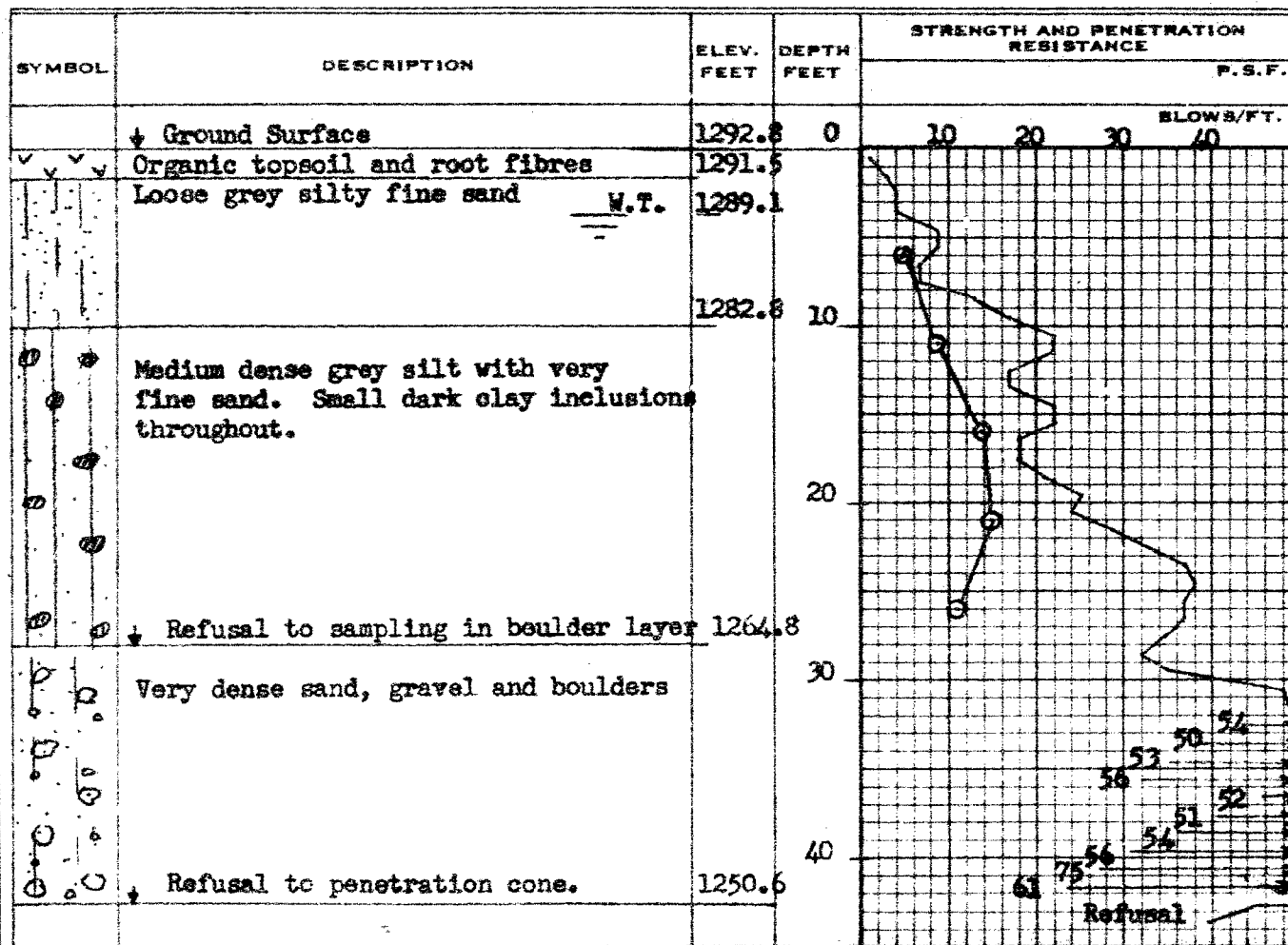
PROJECT White River East Crossing  
 LOCATION Highway #17, Approx. 3 mi. E. of Obrian  
 HOLE LOCATION Chainage 615 + 97, 15' left  
 HOLE ELEVATION AND DATUM 1292.82

BOREHOLE NO. 1  
 FIELD SUPERVISOR L.S.  
 DRILLER M.C.  
 PREP. L.S.

DRAWING NO. 2

## LEGEND

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



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT- % DRY WT.		
	SS1	
	SS2	
	SS3	
	SS4	
	SS5	

PROJECT NO. C 108 J 117

## TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

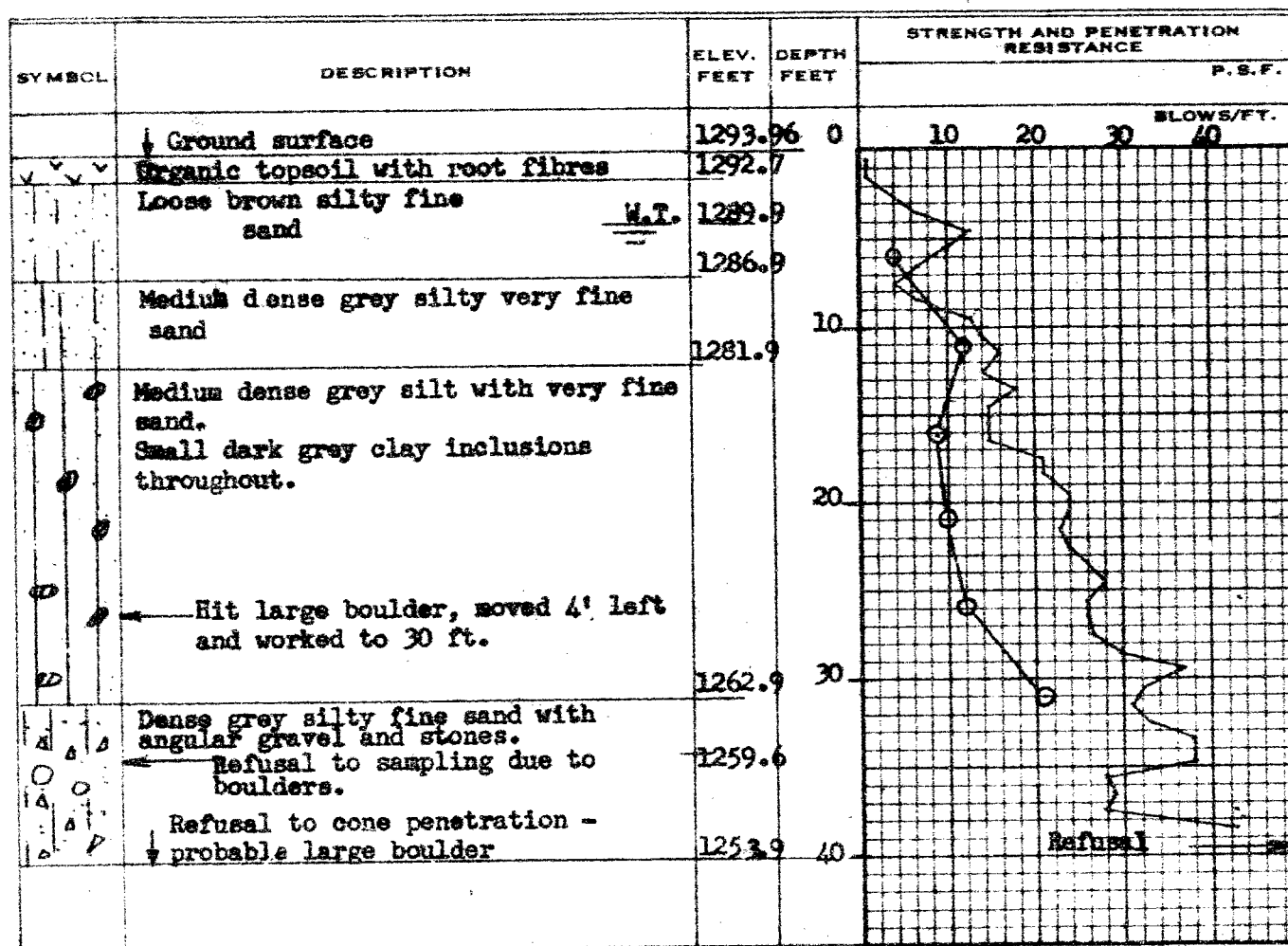
PROJECT White River East Crossing  
 LOCATION Hwy. #17, approx. 3 mi. E. of Obrian  
 HOLE LOCATION Chainage 615 + 97, 15 ft. right  
 HOLE ELEVATION AND DATUM 1293.96

BOREHOLE NO. 2  
 FIELD SUPERVISOR L.S.  
 DRILLER M.C.  
 PREP. L.S.

DRAWING NO. 3

## 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	

PROJECT NO. **C 108 J 117**

**TROW SODERMAN AND ASSOCIATES**

## SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT **White River East Crossing**

LOCATION Hwy. #17, approx. 3 mi. East of Abrian

HOLE LOCATION Chainage 615 + 67 on centre line

HOLE ELEVATION AND DATUM 1293.47

BOREHOLE NO. 3

FIELD SUPERVISOR L.S.

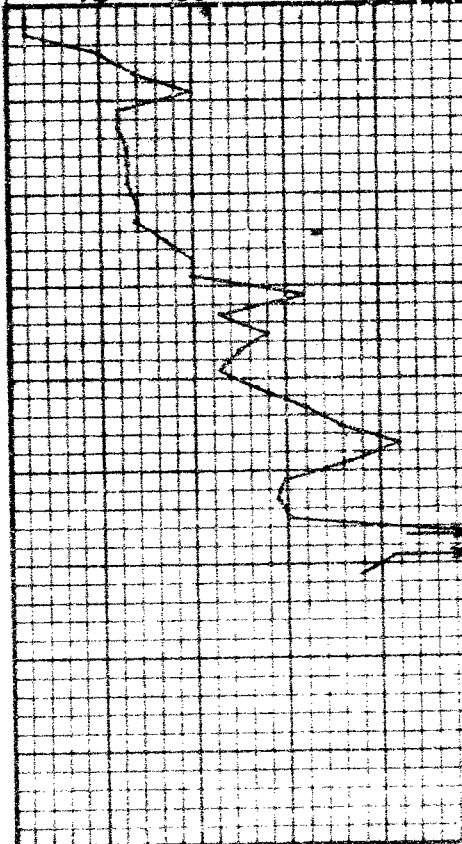
DRILLER M.C.

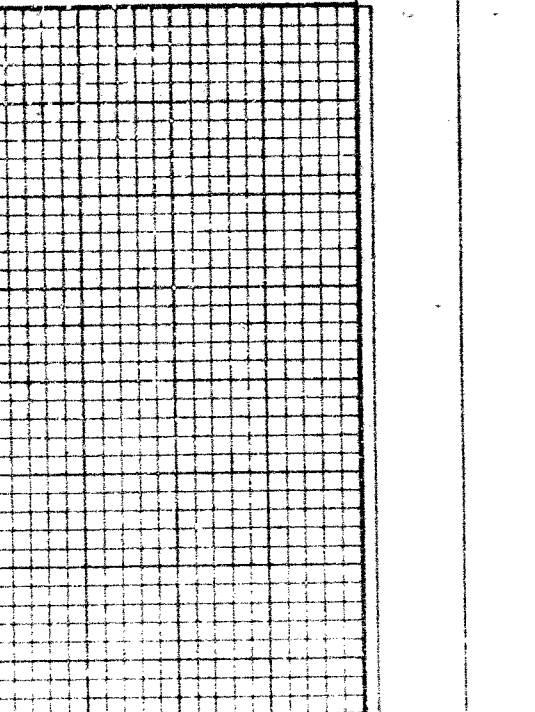
**PREP. L.S.**

DRAWING NO. 4

### LEGEND

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

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE		
				P.S.F.		
				BLOWS/FT.		
	↓ Ground surface	1293.5	0	10	20 30 40	
	Organic topsoil and root fibres	1291.5				
	Probable loose silty fine sand					
		1271.5	10			
	Probable medium dense grey silt with clay inclusions					
			20			
		1265.1				
	↓ Refusal to penetration cone	1263.7	30			
	Probable very dense sand, gravel and boulders					
	NOTE: Above soil horizons based upon dynamic cone test results. Soil types as described are based upon correlation with adjacent borings and cone test.					

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



PROJECT NO. C 108/J117

# TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

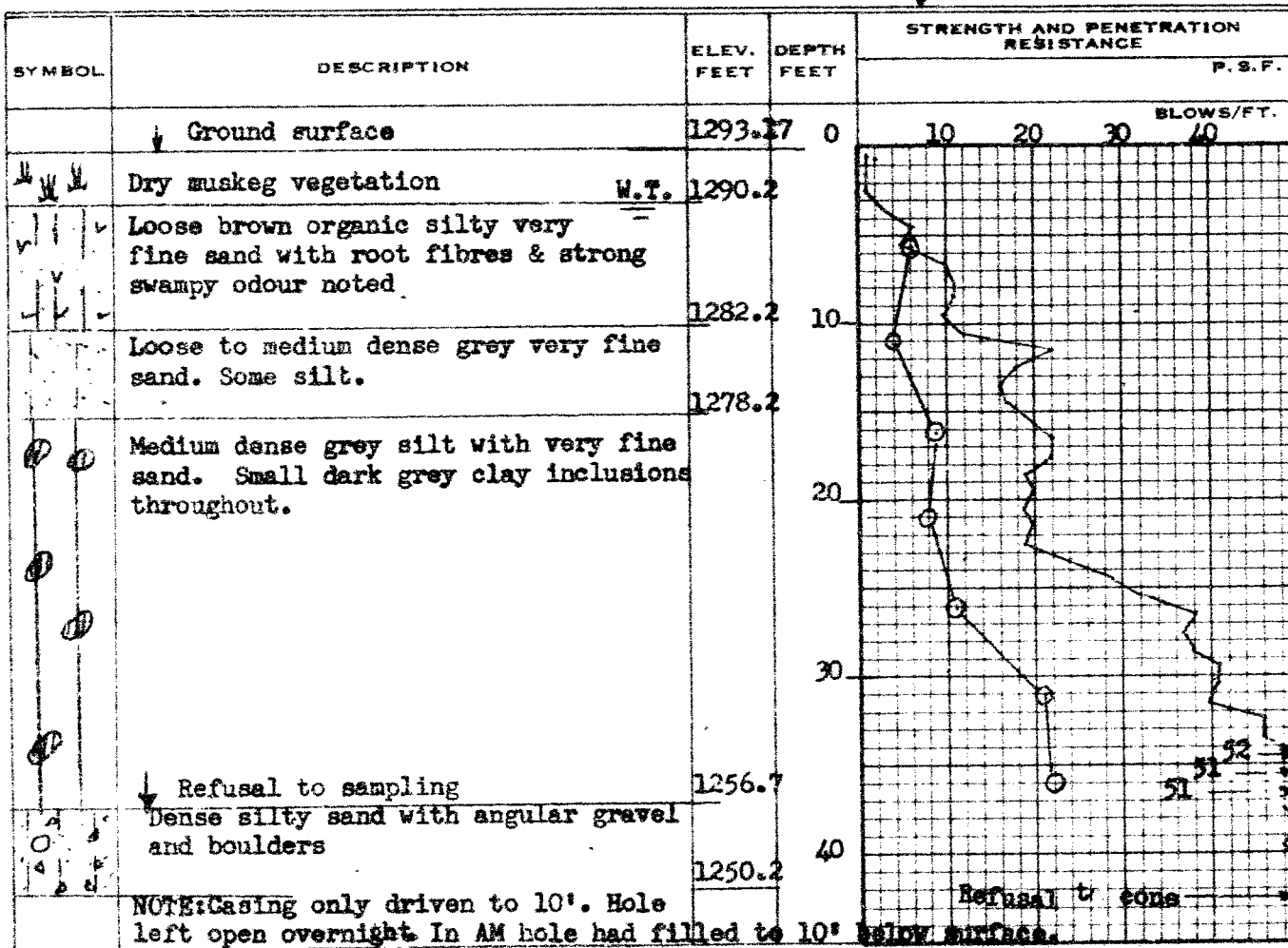
PROJECT White River East Crossing  
 LOCATION Hwy #17, approx. 3 mi. E. of Obrian  
 HOLE LOCATION Chainage 616 + 47, 15 ft. left  
 HOLE ELEVATION AND DATUM 1293.17

BOREHOLE NO. 4  
 FIELD SUPERVISOR L.S.  
 DRILLER M.C.  
 PREP. L.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



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.		
	SS1	
	SS2	
	SS3	
	SS4	
	SS5	
	SS6	
	SS7	

PROJECT NO. 6 108 J 117

# TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

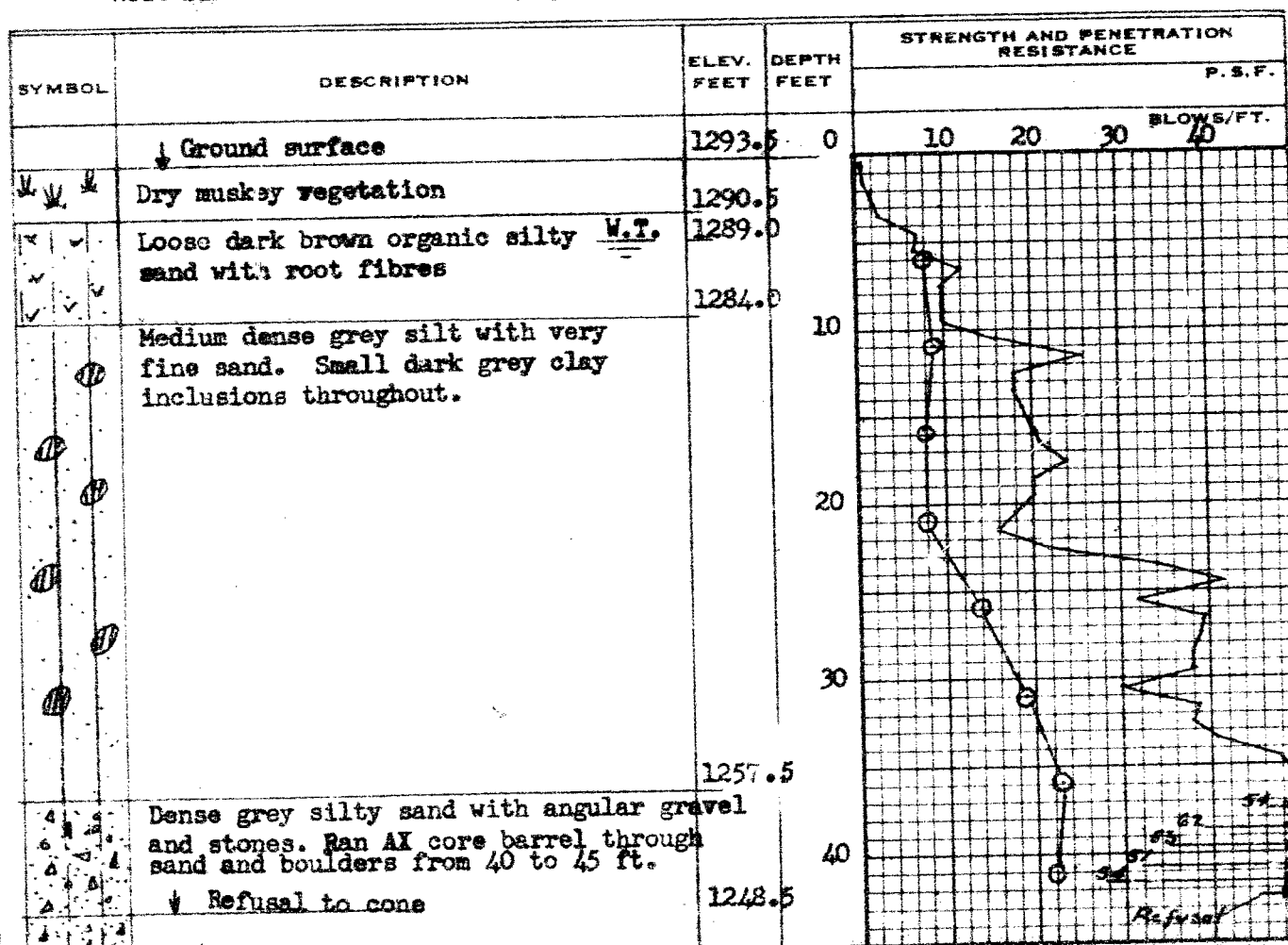
PROJECT White River East Crossing  
 LOCATION Hwy. #17, approx. 3 mi. E. of Obrian  
 HOLE LOCATION Chainage 616+47, 15 ft. right  
 HOLE ELEVATION AND DATUM 1293.47

BORHOLE NO. 5  
 FIELD SUPERVISOR L.S.  
 DRILLER M.C.  
 PREP. L.S.

DRAWING NO. 6

## 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	

PROJECT NO. G 108 J 117

# TROW SODERMAN AND ASSOCIATES

SOIL INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT White River East Crossing  
 LOCATION Hwy. 17, approx. 3 mi. East of Obrian  
 HOLE LOCATION Chainage 616 +77, on centre line  
 HOLE ELEVATION AND DATUM 1293.85

BOREHOLE NO. 6  
 FIELD SUPERVISOR L.S.  
 DRILLER M.C.  
 PREP. L.S.

DRAWING NO. 7

## LEGEND

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

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				P.S.F.	BLOWS/FT.
	↓ Ground surface	1293.9			
↓ ↓ ↓	Dry muskeg vegetation	W.F. 1290.9			
		1290.4			
✓ ✓ ✓	Loose dark brown to black organic silt with fine sand.				
		1282.4			
○ ○ ○	Loose becoming medium dense grey silt with some very fine sand. Small dark grey clay inclusions throughout.				
		1257.4			
△ △ △	Dense silty very fine sand with angular gravel & stones. Refusal to sampling at 43 ft. in gravel & boulders. Refusal to cone at 45 ft.	1240.9			

CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.		
	SS1	
	SS2	
	SS3	
	SS4	
	SS5	
	SS6	
	SS7	
	SS8	