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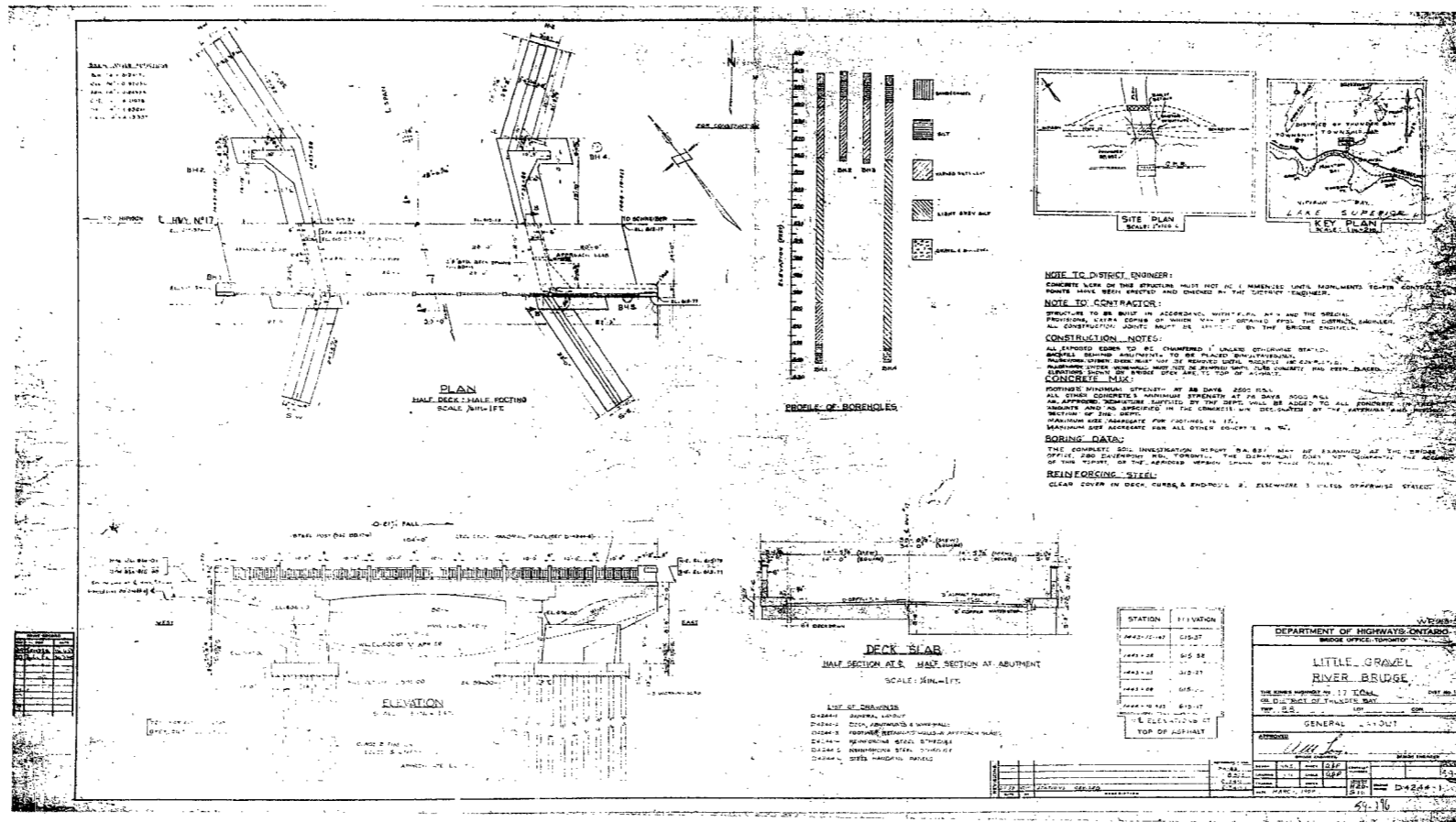
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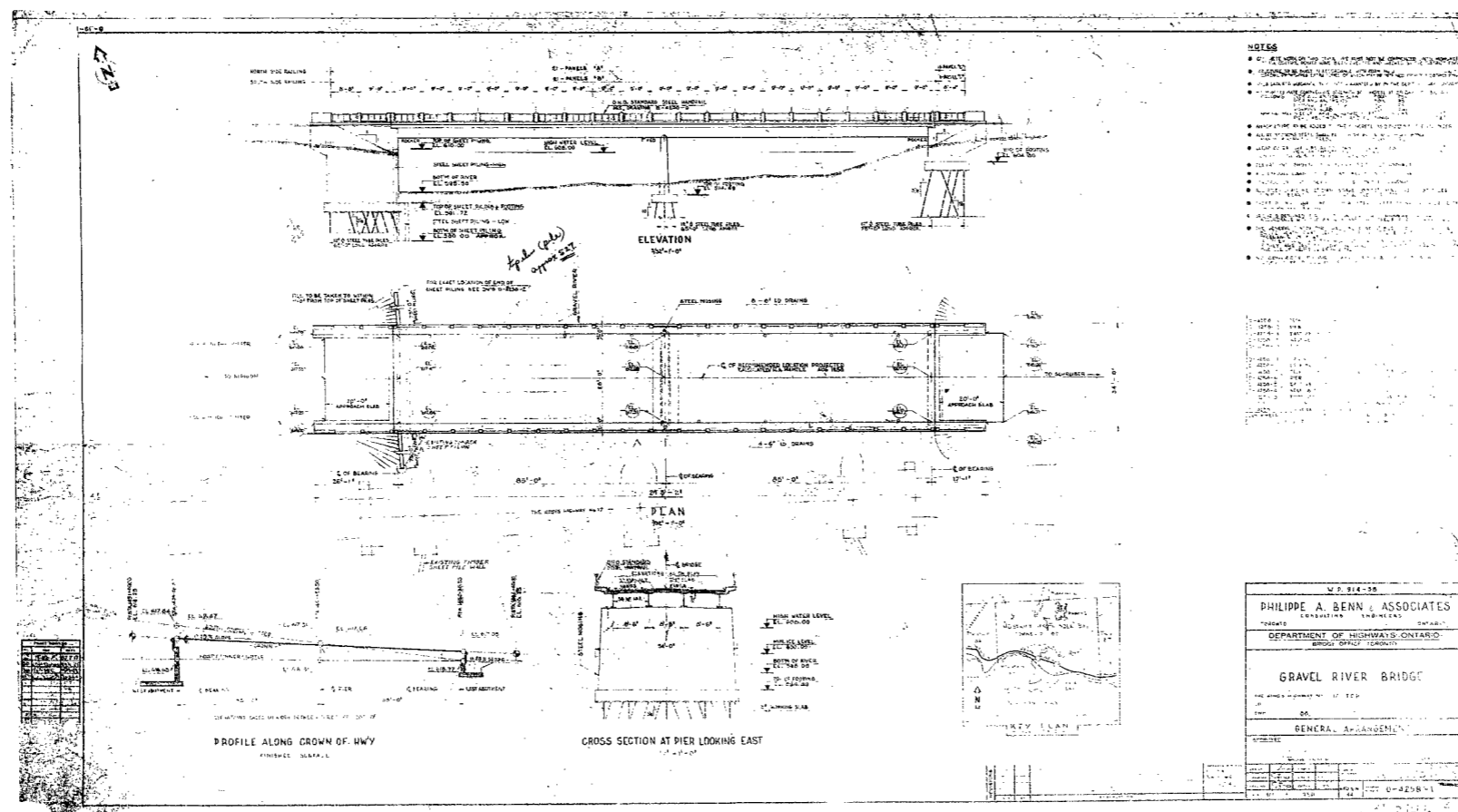
Hwy 17

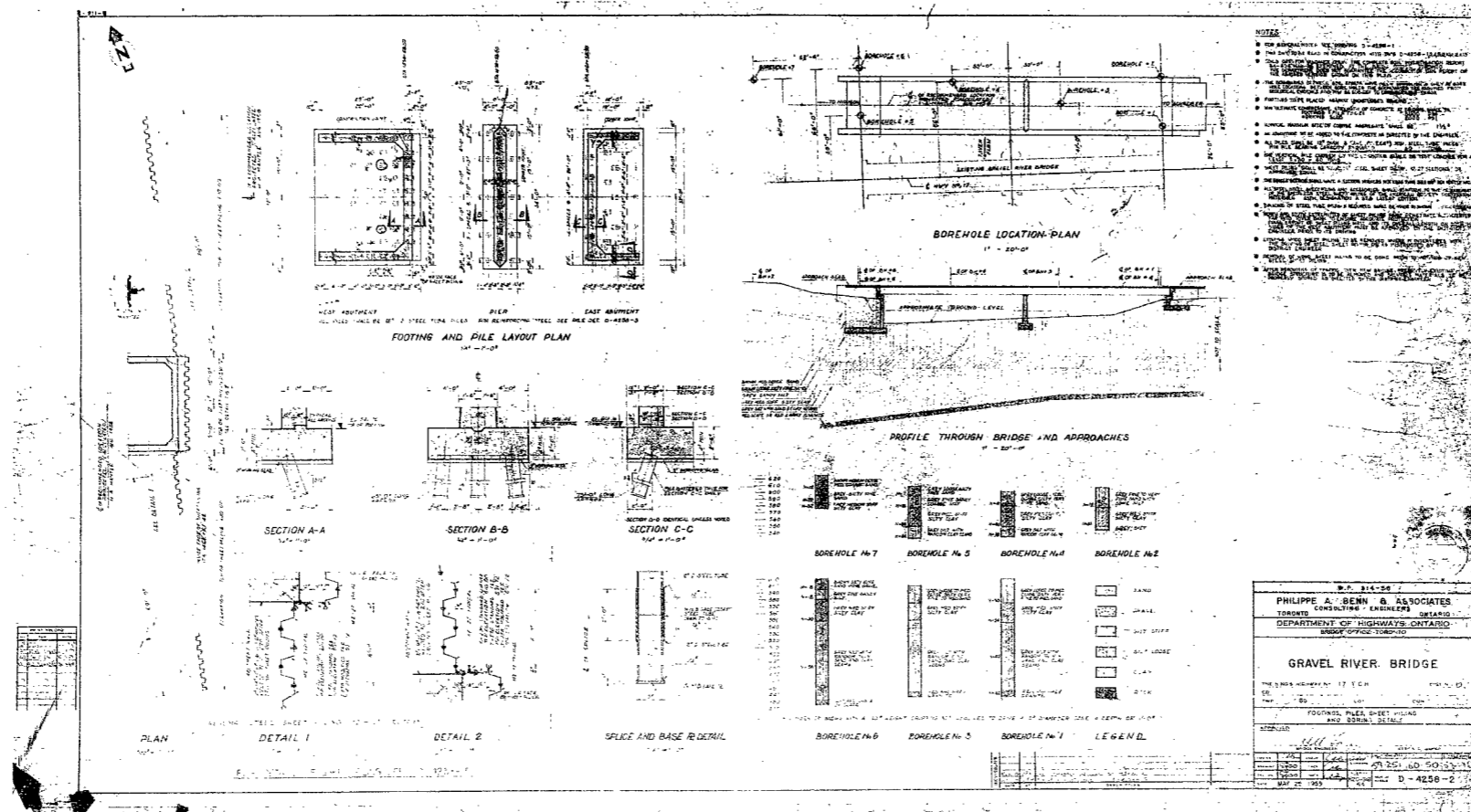
LITTLE GRAVEL

RIVER CROSSING

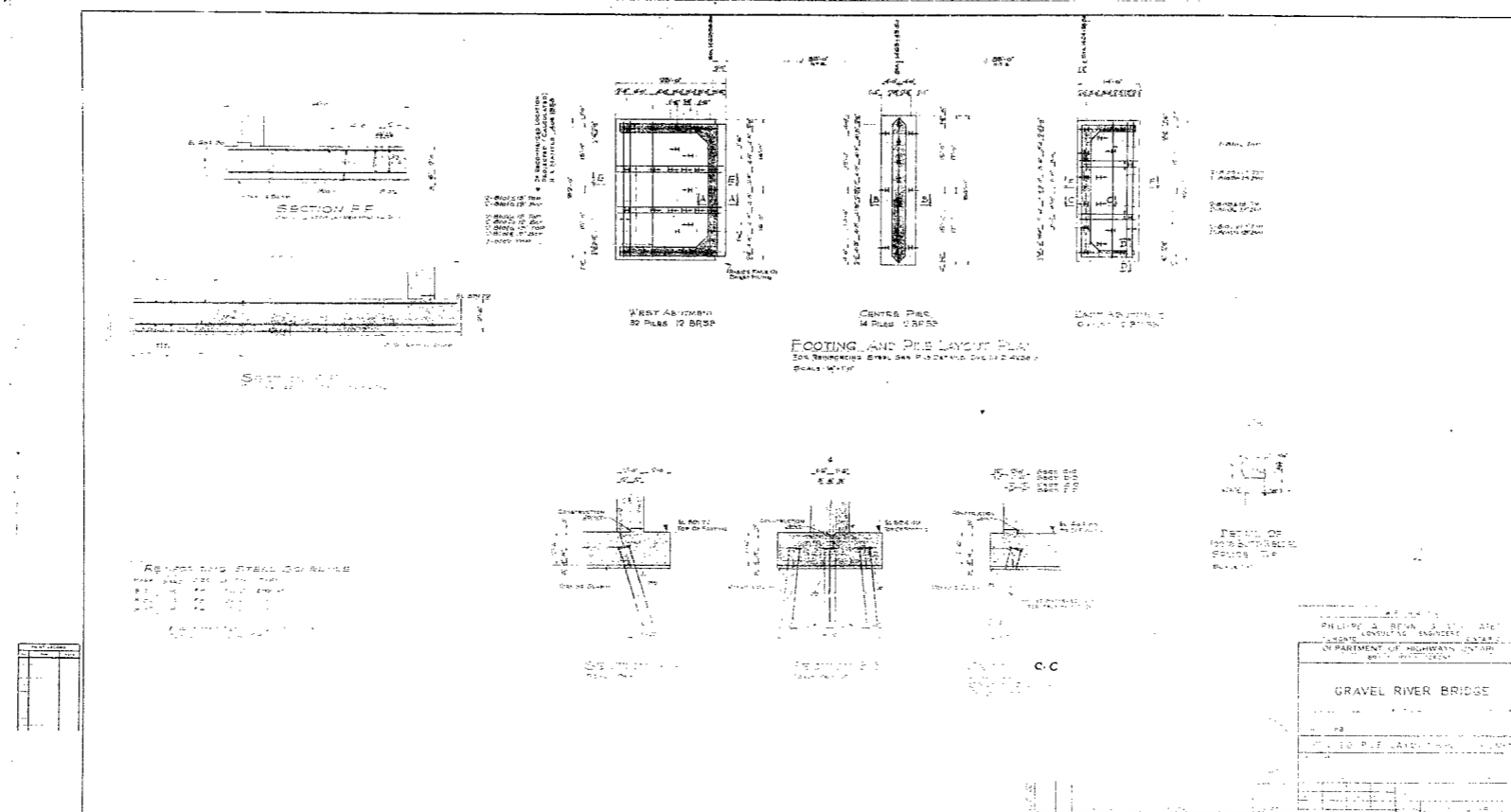


SOME DEFECTS IN NEGATIVE DUE
TO CONDITION OF ORIGINAL DOCUMENTS





SOME DEFECTS IN NEGATIVE DUE
 TO CONDITION OF ORIGINAL DOCUMENTS



BA 837

58-F-269C

TROW, SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

W. A. TROW, M.A.S.C., M.E.I.C., P.ENG.
L. G. SODERMAN, B.S.C., D.I.C., P.ENG.

884 WILSON AVE.,
DOWNSVIEW, ONT.
ST. 8-5921

Project: J267

November 20, 1958

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

Attention Mr. S. McCombie

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

Dear Sirs:

Enclosed herewith is our report on a foundation investigation recently completed at the above noted river crossing. This report not only presents the factual data obtained from the field and laboratory work carried out but also includes our evaluation of the type of pier and abutment support considered practicable and consistent with the strength and compressibility characteristics of the subsoil. For your convenience the principle comments contained in this report are summarized as follows:

- 1) Simple spread footing founded directly upon the upper strata cannot be used to support the pier and abutment loadings at this bridge site.
- 2) Pile supported foundation members are considered necessary. Bearing piles driven to refusal at the dense gravel and boulder layer contacted at a depth below existing ground surface of approximately 165 feet offer a positive means of footing support. An alternative proposal of using comparatively short piles stopped up within the strata of varved clay and/or silt has been presented. Theoretical estimates of pile capacities and settlements resulting from a range of assumed dead load reactions have been made and are tabulated within the body of this report. The results obtained indicate that adequate support can be obtained by using piles typically 40 to 50 feet in length.
- 3) Because of the limitations involved in the analysis of the action of pile groups founded in varved clay deposits and also because of the economy that could be effected through using short

piles it is recommended that a static pile load test be carried out to substantiate the theoretical results presented in this report. In this regard it is pointed out that similar soil conditions exist at two nearby bridge locations and the results of field tests from this site would be of appreciable help in choosing the means of footing support at the nearby river crossings.

4) No problems in connection with embankment foundation instability or overstressing need be anticipated for the fill heights proposed.

We are pleased to have been of service to you on this occasion. If queries come to mind as a result of your review of this report do not hesitate to call upon us for clarification.

Yours very truly,

L.G. Soderman

Lawrence G. Soderman (P. Eng.)

LGS/kb
enc.

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Photographs of Bridge Site	4

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

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

Project: J267

Trow Soderman and Associates

November 20, 1958

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ENCLOSURES

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

Reported herein are the results of a recently completed sub-surface investigation at the above noted site. This report contains the detailed results of field and laboratory findings along with recommendations for foundation type and capacity.

Description of the Site

Little Gravel River crosses Highway No. 17 approximately 29 miles west of Schreiber, Ontario and 93 miles east of Port Arthur, Ontario. The river is presently traversed by means of a 45 foot timber truss bridge and two 15 foot approach spans. The present structure is supported by piles some 45 feet long. The proposed new structure to span the Little Gravel River will be situated on the site of the existing bridge.

The river at the point of crossing is shallow and slow flowing and the depth of water varies from 2 to 3 feet. The shallow river flows into Lake Superior some $\frac{1}{4}$ mile downstream from the bridge site, by way of a relatively straight channel. Little or no erosion appears to have taken place in recent years along the river banks.

The surrounding area is generally flat and covered with a medium stand of birch, poplar and spruce trees. A piled bridge supporting the OPR tracks is located approximately 250 feet downstream from the existing bridge site.

Field and Laboratory Work

During the period of October 6th to 14th four boreholes and their adjacent dynamic cones were driven. The location of these boreholes is indicated on drawing 1. The boreholes were advanced by alternately driving and washing 2-15/32 inch and 3-1/16 inch I.D. casing, with a standard diamond drill adapted for soil sampling.

Boreholes 1 and 4 were advanced until refusal to washing. An attempt to recover a sample of bedrock from borehole 1 resulted in core recovery from several boulders, indicating that a dense layer of gravel and boulders and not bedrock was preventing further penetration. This dense layer was encountered approximately 160 feet from the surface. Boreholes 2 and 3 were advanced until the layer of silt underlying the clay was encountered.

Samples were taken at 5 foot intervals. As the field work progressed and the uniformity of subsoil strata was established the sampling interval was increased. Relatively undisturbed samples were taken in the cohesive material using a 2 inch I.D. Shelby tube sampler. In non-cohesive material samples were taken with a 2 inch O.D. split spoon sampler. The split spoon sampler was driven into the stratum with an energy equal to 350 ft.lbs. per blow i.e. the standard penetration

test. The number of blows (N) required to penetrate one foot are recorded in the summary of laboratory results Table No. 1. When sufficient relatively undisturbed samples were obtained in the cohesive material split spoon samples were taken. In addition to relatively undisturbed samples obtained for shear strength measurements in-situ vane shear tests were performed in the cohesive material. All samples were sealed to retain the original moisture content, and shipped to the Toronto laboratory for testing.

Dynamic cone tests were performed adjacent to each borehole. The dynamic cone profile is presented on each borehole log, drawings 2 to 5. The results of the dynamic cone give a more accurate picture of the density of non-cohesive material than the standard penetration test when such material is located some distance below the water table and when this material is subject to artesian pressures. Where the borehole was not advanced to bedrock dynamic cones were driven in the bottom of the hole. By driving these cones in the bottom of such boreholes, rod friction in the overlying material may be evaluated.

Quick undrained triaxial tests were carried out on the relatively undisturbed samples to give shear strength values. Moisture content, Atterberg limits and unit weight values were determined from various selected samples. In addition to the above tests one consolidation test was performed. The results of the laboratory tests along with a description of each sample taken are summarized in Table No. 3.

All boreholes were referenced to the centre line of the existing bridge. The elevations of the boreholes are referred to the water level indicator found on the east pier.

Soil Types Encountered

A detailed description of the soil types encountered in each borehole, along with the elevations of the upper and lower horizons are included in the borehole logs, drawings 2 to 5. In addition to the detailed description of soil types a cross section estimating the sub-soil stratigraphy has been presented as drawing 1.

Brown Sand and Gravel Fill. This material exists from the surface to a depth of approximately 5 feet. The sand and gravel fill provides an access ramp to the recently placed reinforced Bailey Bridge, as well as serving as base coarse for the existing highway. This material is in a loose state with N values ranging from 2 to 10.

Grey Fine to Coarse Silt. Immediately underlying the fill material is a loose deposit of silt. This material varies in grain size from a very fine sand to a fine silt with the finer material generally appearing near the lower extremity of the deposit. The silt layer is some 8 to 10 feet in depth. Average values of N for this stratum range from 2 to 3 indicating its loose nature.

Varved Silty Clay. A deposit of varved silty clay was found in all boreholes immediately below the previously described silt. The depth of this stratum is approximately 35 feet. The material may be described as soft since laboratory and in-situ field vane measurements of strength indicate cohesive values ranging from 300 to 600 psf. An average of all strength values in the clay gives a value of 450 psf.

Moisture content tests were carried out on both composite samples made up of the clay and silt phase of a varve, and on the clay phase of a varve. Results of the composite moisture tests for samples taken at a depth of 30 feet from 3 boreholes give values ranging from 52.5 to 53.1. The small variation in moisture content indicates the uniformity of the varved silty clay over the site investigated. Moisture contents of the clay phase of a varve also confirm the uniformity of the deposit since these values are all in the 70 percent range. The composite moisture content lies approximately midway between the plastic limit of 22.5 and the liquid limit of 67 percent. The unit weight of the varved silty clay varies from 97 to 107 psf.

Consolidation characteristics for the clay stratum have been estimated from the results of consolidation tests on samples of similar varved silty clay. The varved silty clay at Little Gravel River bridge site is similar (on the basis of laboratory results) to two other bridges investigated in the near vicinity. On this basis a value of 0.6 has been selected for the compression index C_c .

In addition to the stratum of silty clay described above a deposit of stiffer but similar material was encountered in borehole 1 at a depth of 135 feet below ground surface. Because of the extreme depth and dense nature of this material, settlements in this stratum resulting from piles driven from the surface some 50 to 60 feet will be negligible. A value of the compression index C_c obtained from consolidation tests in this material is 0.44.

Light Grey Silt. The stratum of light grey silt extends from the bottom horizon of the silty clay some 100 feet to the top horizon of the gravel and boulders. It exists in a medium dense state with N values ranging from 20 to 40. Thin layers of fine sand and of clay are found at random depths throughout the deposit. These layers are sufficiently wide spread so as to have no influence on the silt characteristics of the material.

Gravel and Boulders. A dense layer of gravel and boulders was encountered 165 feet below ground surface. Refusal to further penetration of the dynamic cone or wash rods was experienced at the top of this stratum. The material was not sampled by conventional methods (i.e. split spoon) but in borehole 1 coring with a diamond bit recovered several pieces of core from various boulders.

Foundation Considerations

Due to the loose nature of the upper layer of fine to coarse silt, spread footings founded directly upon this stratum cannot be used. The use of end bearing piles founded on the dense layer of gravel and boulders appears uneconomic (150 ft length required). For these reasons the practical solution would be the use of large displacement piles supporting their loads by a combination of skin friction and end bearing. These large displacement piles may be terminated in the clay stratum or driven into the underlying silt deposit. The length of pile selected is dependent upon the total design load carried by the abutments. These large displacement piles will not be driven to any required "set" but are simply driven to the tip elevation required to give the needed load capacity. The maximum loads which various diameter piles are capable of carrying along with the corresponding pile tip elevations are presented in table No. 1.

From laboratory tests measuring the shear strength and vane in-situ shear strengths the average value of 450 psf has been assigned to the cohesion of the varved silty clay. This value of 450 psf may be used in the calculations of bearing capacities and end bearing of piles in the clay, but is not the true value for the calculation of skin friction on piles. In accordance with recent research published by M.J. Tomlinson in the "Fourth International Conference on Soil Mechanics 1957" a value of adhesion equal to 0.8 of the true value is recommended. For this reason a value of 360 psf has been used in the calculation of skin friction of the piles.

Pile capacities have been calculated for pile tip elevations of 550, 560 and 570 feet. If the pile cut-off elevation is just below river level (600) the resulting pile lengths would be 50, 40 and 30 feet. Piles terminating at elevation 560 and 570 will be entirely surrounded by clay. These piles will depend mainly on skin friction for their load carrying ability. Piles driven to a tip elevation of 550 feet will have end bearing in the silt. The end bearing in non-cohesive type material such as this silt is much higher than in the overlying clay material, thus these piles will have a much higher capacity than the previously described piles.

For piles driven to elevations 550 and 560

$$\text{maximum load} = \frac{A_s \times C_c}{F.S.} + \frac{C N_c}{F.S.}$$

where A_s = area of pile surface subject to skin friction.

C_c = cohesive strength corrected for adhesion
= 360 psf.

F.S. = factor of safety.

C = cohesive strength = 450 psf

N_c = bearing capacity factor = 9.

If we consider a 12 inch diameter pile driven to elevation 560, and assume the top of the clay stratum at elevation 594, the surface area subject to skin friction will be 3.14×34 sq.ft. and

$$\text{maximum load per pile} = \frac{3.14 \times 34 \times 360}{2000 \times 3} + \frac{9 \times 450 \times .78}{2000 \times 3} = \underline{6.9 \text{ tons.}}$$

For the same 12 inch diameter pile driven into the silt with pile tip elevation 550 the skin friction would be the same as previously calculated (skin friction in the silt is neglected). The end bearing in the silt is calculated from information given by Meyerhof A.S.C.E. January 1956

$$\text{end bearing} = \frac{4 N A_p}{F.S.} \text{ tons}$$

where N = standard penetration resistance (number of blows) per foot penetration in vicinity of pile tip = 15.

A_p = sectional area of pile.

$$\text{Pile Capacity} = \frac{3.14 \times 34 \times 360}{2000 \times 3} + \frac{4 \times 15 \times .78}{3} = 22 \text{ tons.}$$

By similar calculations the following tabulation has been made:

Pile Tip Elev. Ft.	Pile Diameter Inches	Max. Load Per Pile Tons
570	8	3.2
570	10	4.1
570	12	5.0
560	8	4.5
560	10	5.7
560	12	6.9
550	8	11.2
550	10	16.1
550	12	22.0

Settlements resulting from estimated bridge dead loads have been calculated using the compression indices of 0.51 and 0.2 for the clay and silt respectively. The settlements for various dead loads with piles at different tip elevations are presented in Table No. 2. In making these calculations it has been assumed that piles at elevations 560 and 570 will be placed in three rows of 10 piles each with a centre to centre distance of 3 feet, this results in a pier or abutment size approximately 30 x 6ft. With piles at tip elevation 550 20 piles can be used placed in 2 rows of 10 piles with centre to centre spacing of 3 feet giving an abutment size approximately 30 x 4 ft. The load carried by these piles may be considered to dissipate at a slope of 2 : 1 from the lower 1/3 point of the pile. In the calculation of the value p_0 the original overburden pressure,

it was assumed that the original ground elevation was 605 feet. The value of settlement (S) for any increment of depth may be found from

$$S = \frac{H \times C_c}{1 + e_o} \log \frac{P_o \times \Delta p}{P_o}$$

where H = thickness of layer considered.

C_c = compression index (clay = .6, silt = .2)

e_o = initial void ratio (clay = 1.6, silt = 0.95)

P_o = original effective overburden pressure at the centre of the layer considered.

Δp = change in effective pressure at the centre of layer considered due to deadload on piles.

By inserting appropriate values for the assumed dead load on the piles the results may be summarized as follows:

TABLE No. 2

<u>Pile Tip Elevation</u> <u>Feet</u>	<u>Max. Dead Load</u> <u>Tons</u>	<u>Settlement</u> <u>Inches</u>
570	100	6.0
570	150	9.1
560	150	7.6
560	200	8.9
550	200	5.4
550	300	7.4

The best selection of pile length and thus resulting settlement is dependent of the bridge loading. Assuming that the combination of live and dead loads will be greater than 200 tons the choice of piles would be a pile of 8, 10 or 12 inches in diameter driven to elevation 550. It is probable that 10 inch wooden piles 50 feet long will be more easily obtained than larger diameter, thus 20 piles will have a capacity of $20 \times 16 = 320$ tons. The settlement of the pile group is dependent on the dead weight on the group of piles. Table No. 2 gives a value of approx. 5 inches for a pile group carrying 200 tons dead load and having pile tip elevation at 550 feet.

Settlements of piles driven into the silt should have neared completion by the end of construction of the bridge. Piles terminating in the clay will continue to settle gradually with the settlement stated in table 2 reaching 90% completion in 6 to 8 years.

Summary

The loose state of the upper granular layer and the deep underlying cohesive strata existing at this site indicate that a floating pile foundation consisting of large displacement piles would be the most

satisfactory means of footing support. These large displacement piles will support load by a combination of skin friction and end bearing. Pile length and diameter may be selected from Table No. 1 once the design loadings are known.

Settlements calculated for a range of dead loading have been presented on Table No. 2. For the probable dead loads to the piers and abutments of this structure it appears that settlements are within tolerable limits.

Piles driven to tip elevations consistent with calculated safe loads per pile tabulated in the body of this report will not meet refusal to driving.

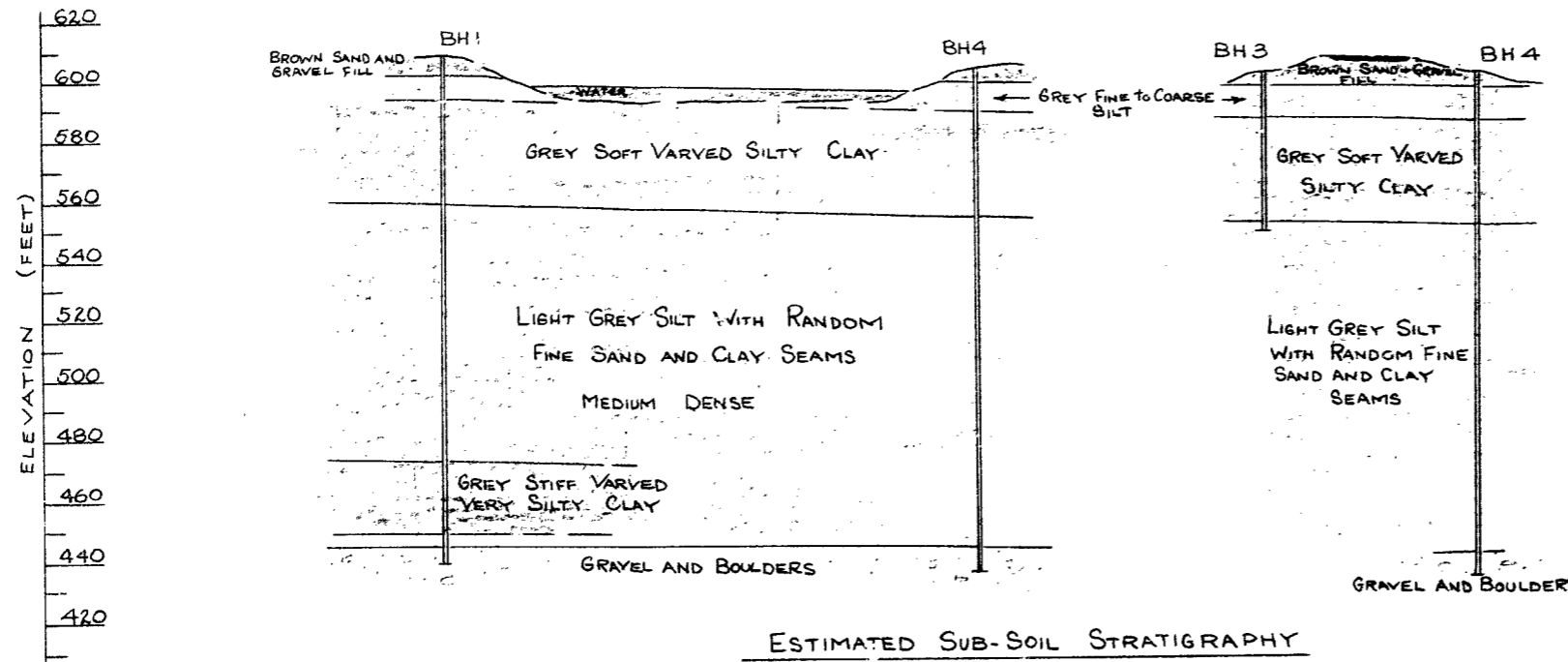
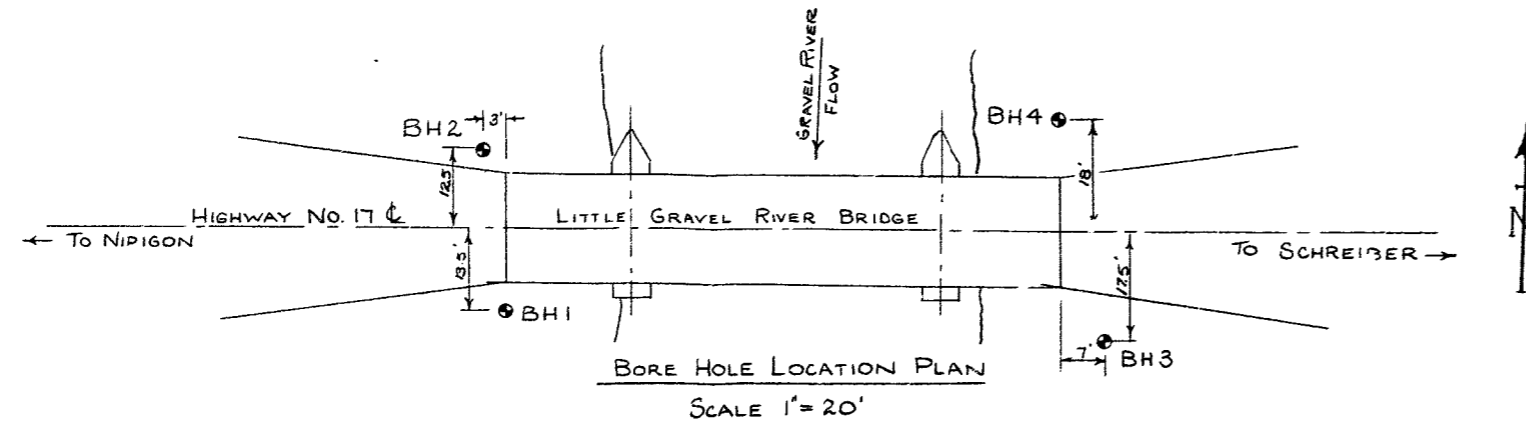
No problems in connection with embankment instability are considered possible with the low fill heights proposed.

H. Trow for

KF/kb

Kenneth R. Peaker (P. Eng.)

DRAWING No. 1
JOB No. C108/J267



LITTLE GRAVEL RIVER

BORE HOLE LOCATION
AND
SUB-SOIL STRATIGRAPHY

SCALE: AS SHOWN
OWN BY: K. P.
CHKD. BY: L. G. S.

Nov. 1958

PROJECT NO. C108/J267

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Little Gravel River Bridge

LOCATION Highway No. 17

HOLE LOCATION See plan

HOLE ELEVATION AND DATUM 609.8

BOREHOLE NO. 1

FIELD SUPERVISOR K.P.

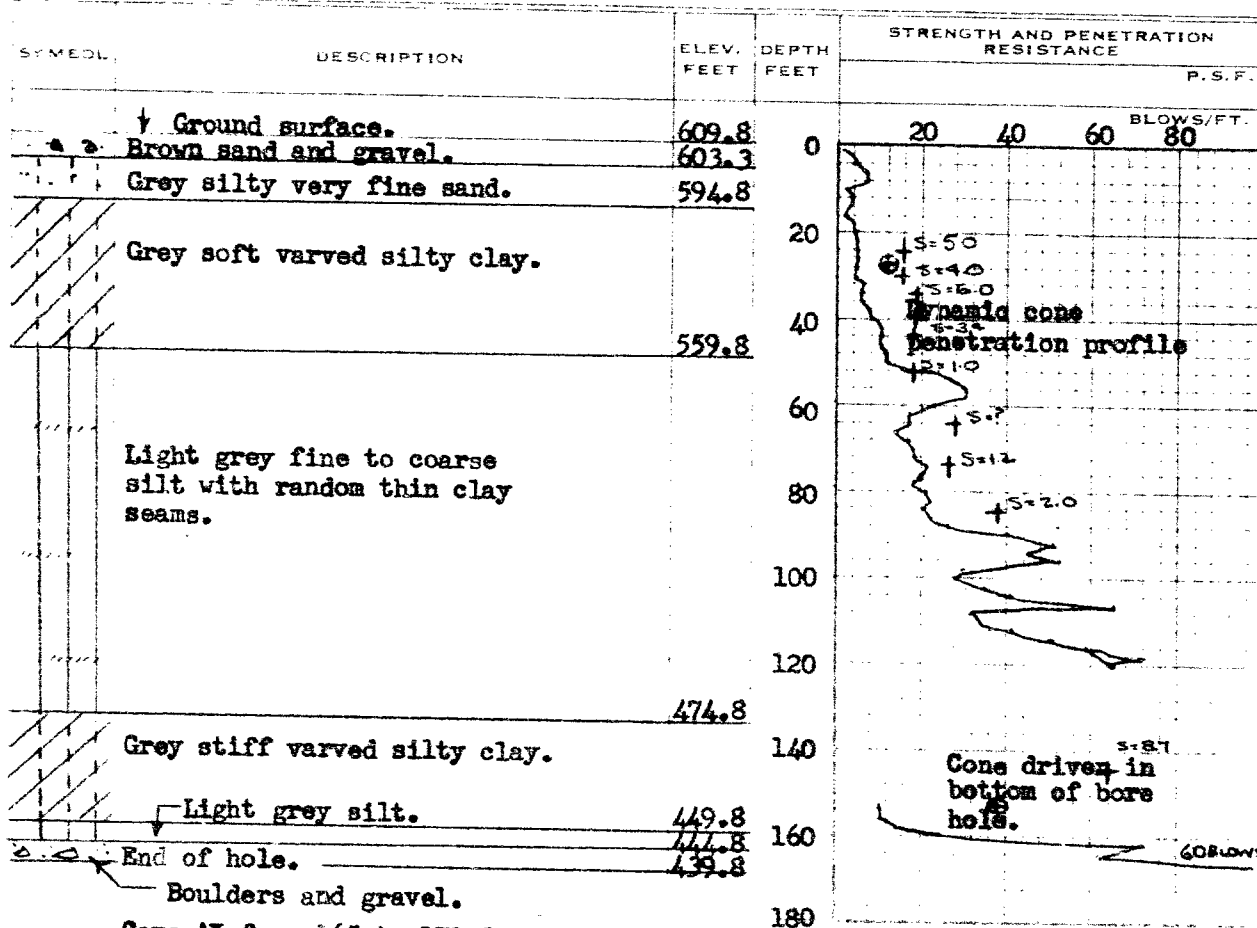
DRILLER H.U.

PREP. K.P.

DRAWING NO. 2

LEGEND

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



CONSISTENCY		SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.			
		SS1	
		SS2	
		SS3	
		SS4	
		TW5	
		SS6	
		SS7	
		SS8	
		SS9	
		SS10	
		SS11	
		SS12	
		SS13	
		SS14	
		SS15	
		SS16	
		SS17	
		SS18	
		SS19	
		TW20	

Ground water level = 0.5 ft in caved hole.

PROJECT NO. **C108/J267**

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

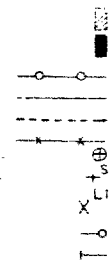
PROJECT **Little Gravel River Bridge**
 LOCATION **Highway No. 17**
 HOLE LOCATION **See plan**
 HOLE ELEVATION AND DATUM **610.3**

BOREHOLE NO. **2**
 FIELD SUPERVISOR **K.P.**
 DRILLER **H.U.**
 PREP. **K.P.**

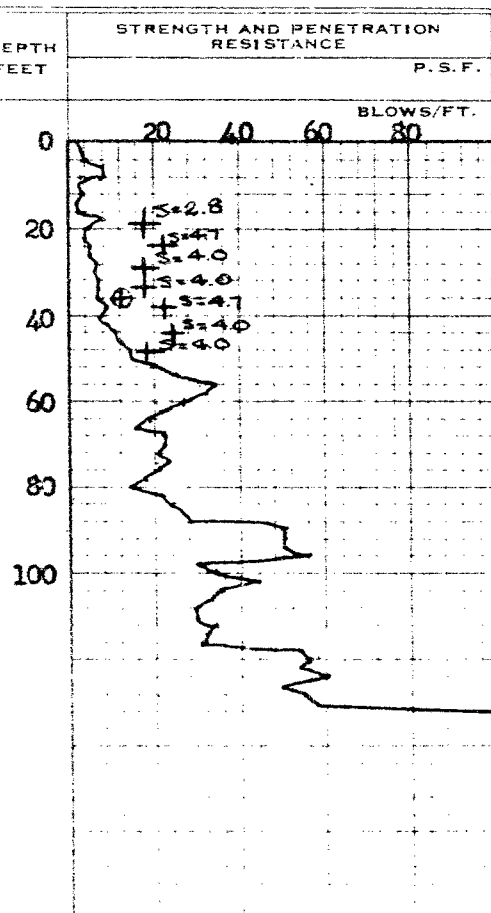
DRAWING NO. **3**

LEGEND

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



SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
↓	Ground surface.	610.3	0
	Sand and gravel fill.	605.3	
	Grey silty very fine sand.	595.3	
	Grey soft varved silty clay.		
		560.3	
↙	End of hole.	558.3	
	Light grey silt with random thin clay seams.		
	Ground water level = 3 ft.		



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.		
	SS91	
	SS92	
	SS93	
	SS94	
	SS95	
	SS96	
	SS97	
	SS98	
	SS99	
	SS100	

PROJECT NO. C108/J267

TROW SODERMAN AND ASSOCIATES

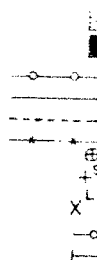
ITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

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



PROJECT Little Gravel River

LOCATION Highway No. 17

HOLE LOCATION See plan

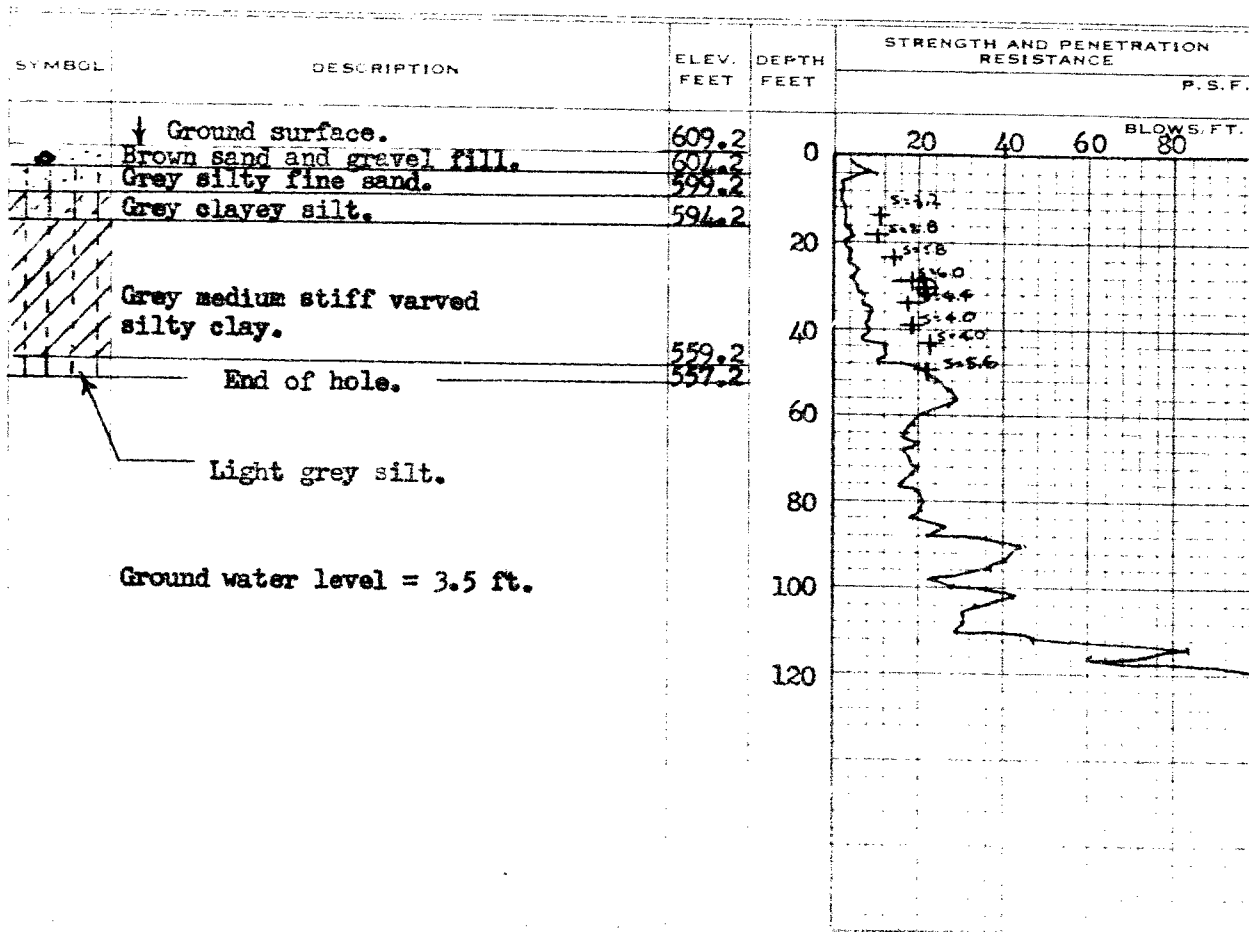
HOLE ELEVATION AND DATUM 609.2

BOREHOLE NO. 3

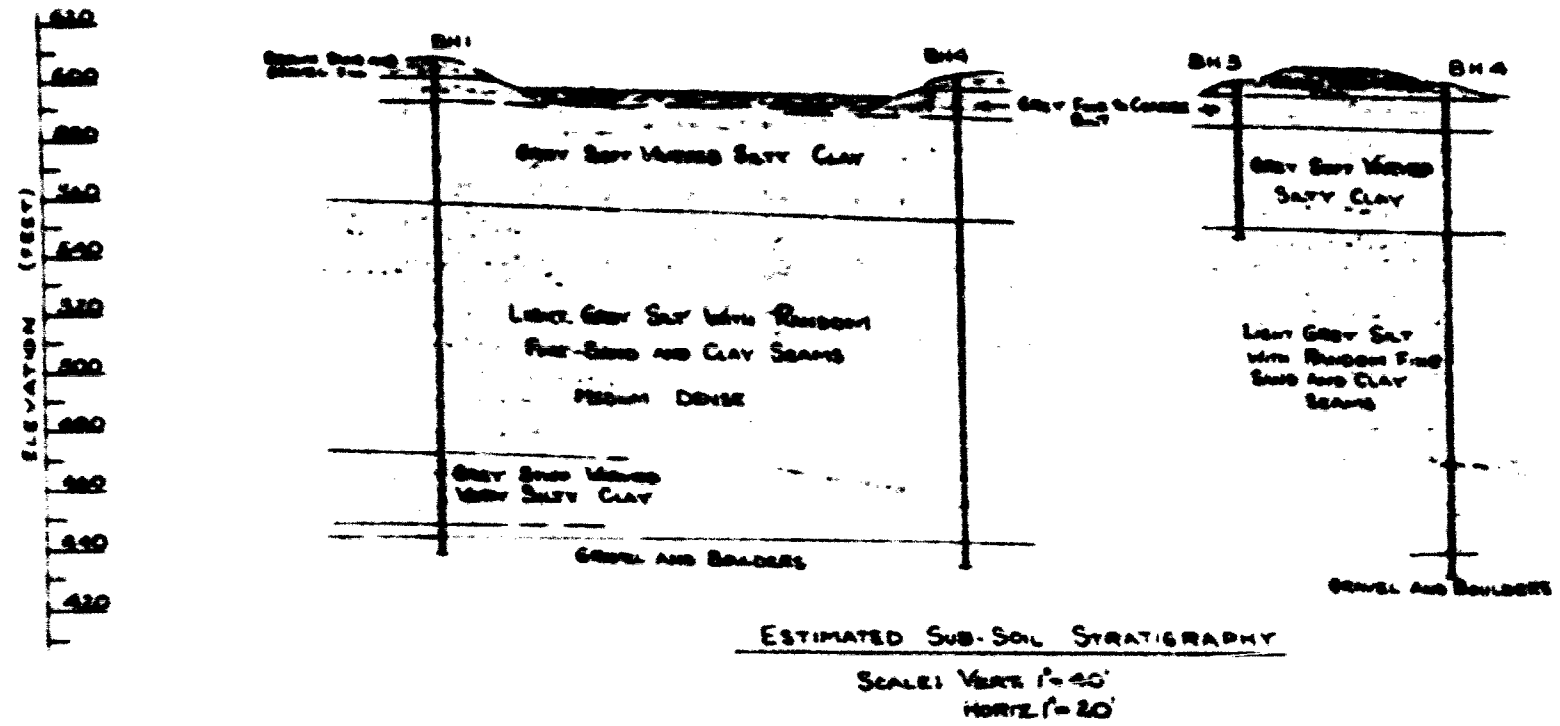
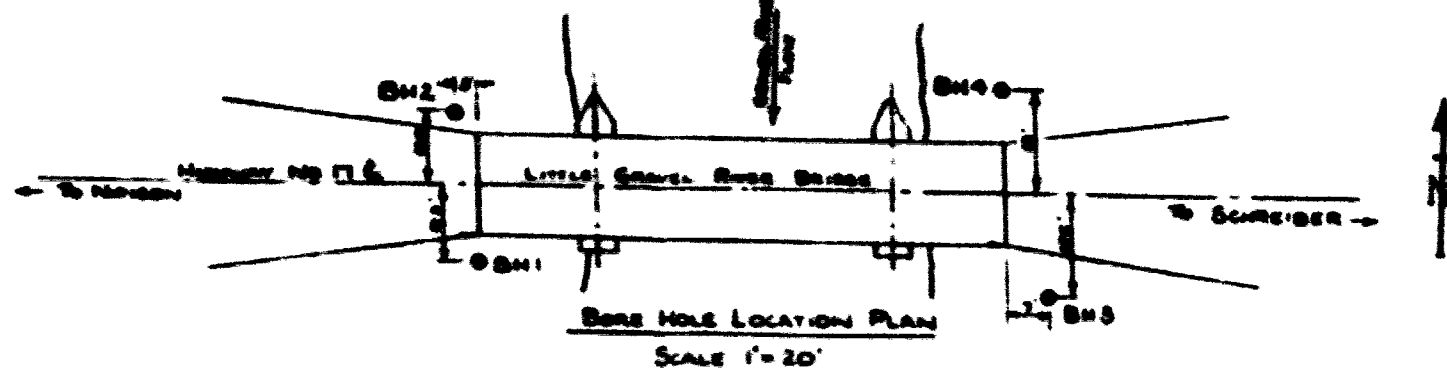
FIELD SUPERVISOR K.P.

DRILLER E.S.

PREP. K.P.



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT- % DRY WT.		
	SS1	
	SS2	
	SS3	
	SS4	
	SS5	
	SS6	
	SS7	
	SS8	
	SS9	
	SS10	



LITTLE GRAVEL RIVER
BORE HOLE LOCATION
 AND
SUB-SOIL STRATIGRAPHY

SCALE: AS SHOWN
 DWN BY: K. P.
 CHKD BY: L. G. S.

TROW SODERMAN AND ASSOCIATES

INVESTIGATION AND SOIL MECHANICS CONSULTATION

Little Gravel River

Highway No. 17

HOLE NO. 1 See plan

ELEVATION OF SURFACE 609.2

BENCHMARK NO. 3

FIELD SUPERVISOR L.P.

DRILLER R.3.

PREP. L.P.

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

SYMBOLS USED HEREIN

ELEV. DEPTH
FEET FEETSTRENGTH AND PENETRATION
RESISTANCE

P.S.F.

CONSISTENCY

NATURAL

SAMPLE UNIT WT

P.C.B.

MOIST. CONTENT - % DRY WT

20 40 60

BLOWS FT

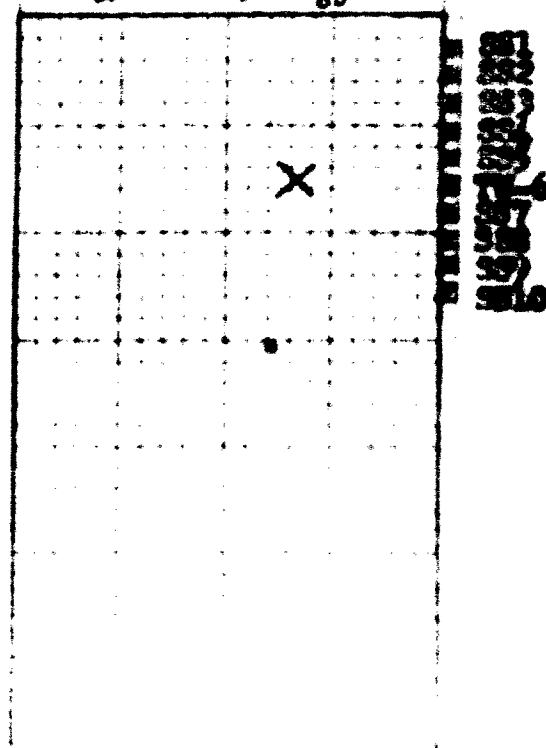
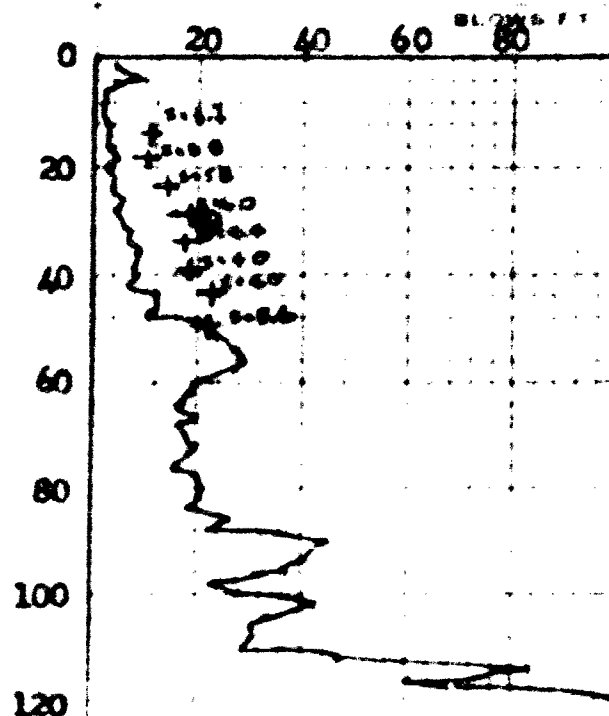
Ground surface. 609.2
 Brown sand and gravel fill. 607.2
 Gray silty fine sand. 599.2
 Gray clayey silt. 594.2

Gray medium stiff varved
 silty clay. 559.2

End of hole. 537.2

Light gray silt.

Ground water level = 3.5 ft.



PROJECT NO. C108/J267

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

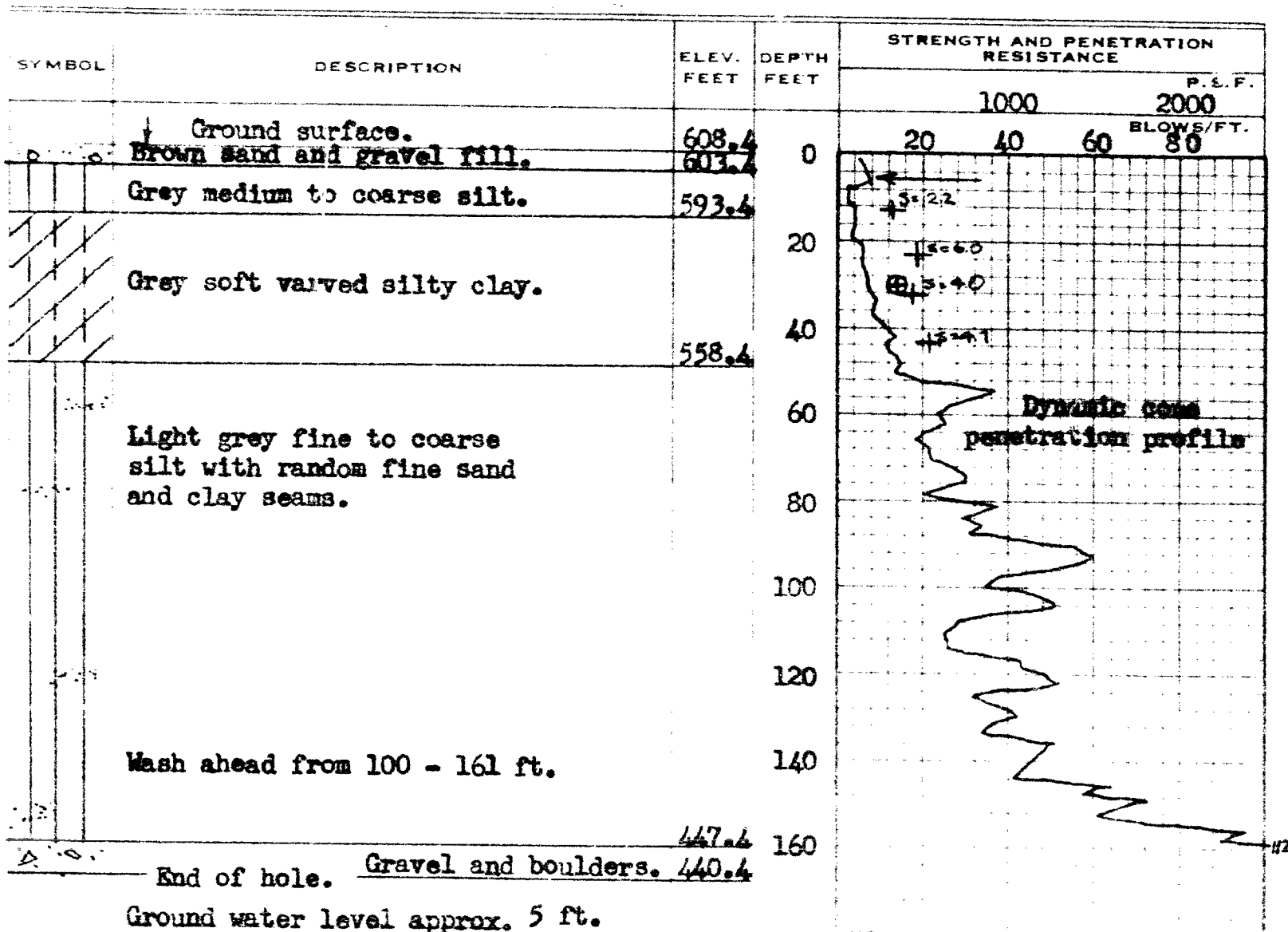
PROJECT **Little Gravel River**
 LOCATION **Highway No. 17**
 HOLE LOCATION **See plan**
 HOLE ELEVATION AND DATUM **608.4**

BOREHOLE NO. **4**
 FIELD SUPERVISOR **K.P.**
 DRILLER **E.S.**
 PREP. **K.P.**

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	
	TW3	
	SS4	
	SS5	
	SS6	
	SS7	
	SS8	
	SS9	
	SS10	

Ground water level approx. 5 ft.

Uncorrected Laboratory Pressure Void Ratio Curve.

Borehole 1, Sample 19, Sample Depth 150 - 151½ ft.

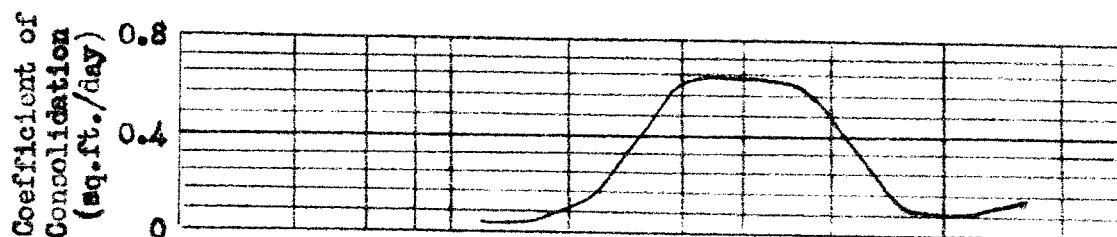
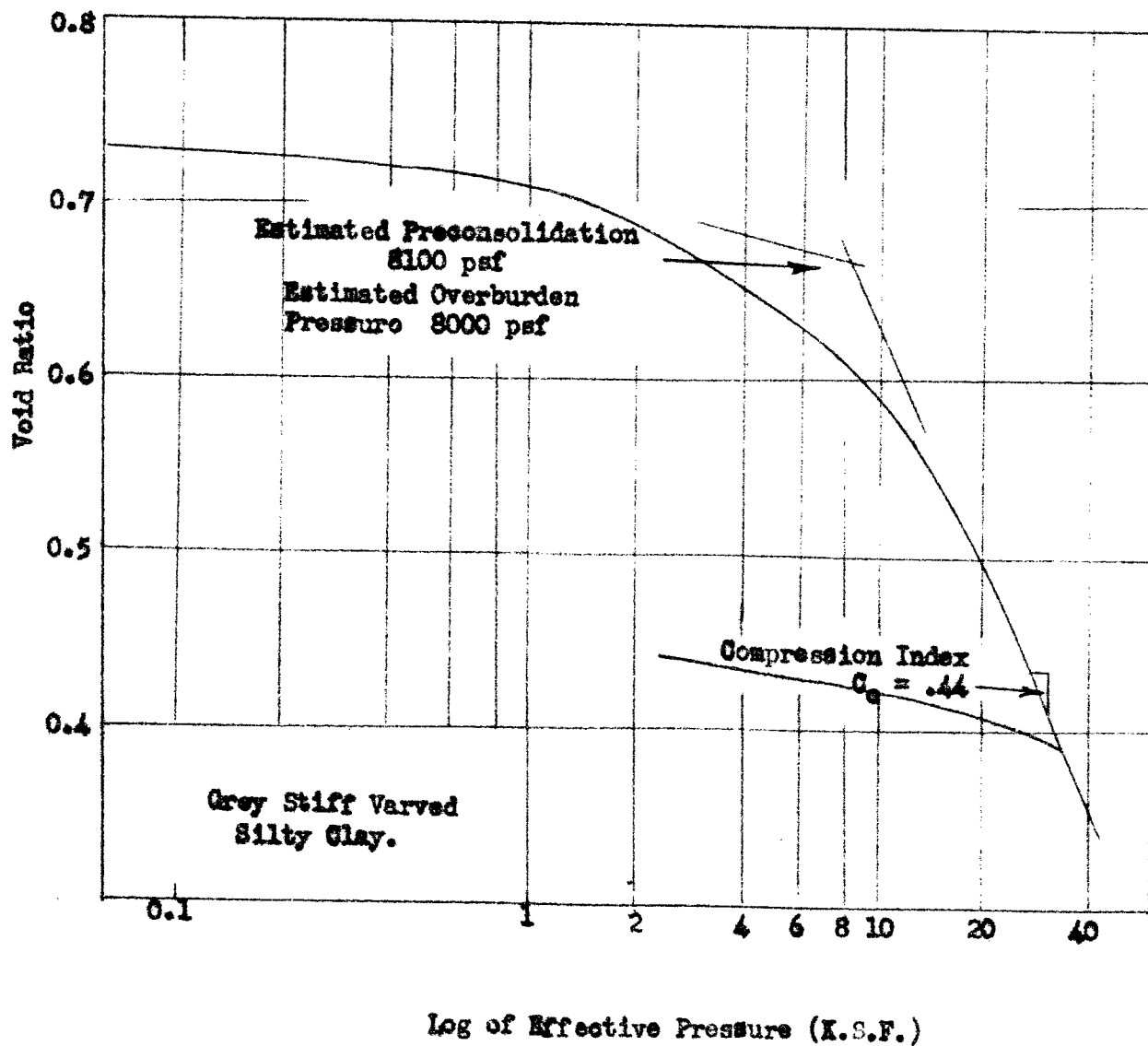


TABLE No. 3
SUMMARY OF LABORATORY TEST RESULTS

Hole No.	Sample No.	Depth Ft.	Description	Shear Strength		Consistency			Unit Wt. p.s.f.	N
				p. s. f. Vane*	C.	% dry wt. W	L.L.	P.L.		
1	SS1	5 - 7	Brown fine to coarse sand & gravel.							P
	SS2	10 - 12	Grey silty fine sand.							P
	SS3	15 - 17	Grey varved silty clay.							P
	SS4	20 - 22	Grey varved silty clay.	380						P
	TW5	25 - 26½	Grey varved silty clay.	375	290	72.5+			97.2	P
	SS6	30 - 32	Grey varved silty clay.	460		60.4				P
	SS7	35 - 37	Grey varved silty clay.	446						P
	SS8	40 - 42	Grey varved silty clay.	446						P
	SS9	50 - 52	Grey varved clayey silt.	420						P
	SS10	60 - 62	Grey slightly clayey silt.	676						P
	SS11	70 - 72	Grey fine to coarse grained silt.	625						P
	SS12	80 - 82	Grey slightly clayey silt.	970						P
	SS13	90 - 92	Grey coarse silt.							P
	SS14	100 - 102	Grey fine sandy silt.							P
	SS15	110 - 112	Grey sandy silt.							4
	SS16	120 - 122	Grey very fine sand.							24
	SS17	130 - 132	Grey clayey silt.							14
	SS18	140 - 142	Grey varved silty clay.	1580						P
	TW19	150 - 151½	Grey varved silty clay.	2900	960	32.1			120.8	P
2	SS1	5 - 7	No recovery - grey silty sand.							7
	SS2	10 - 12	Grey medium grained silt.							P
	SS3	15 - 17	Grey clayey silt.	440						P
	SS4	20 - 22	Grey varved silty clay.	530						P
	SS5	25 - 27	Grey varved silty clay.	445						P
	SS6	30 - 32	Grey varved silty clay.	445						P
	TW7	35 - 36½	Grey varved silty clay.	530	300	67.0+	67.0	22.5	105.2	P
	SS8	40 - 42	Grey varved silty clay.	600		52.5				P
	SS9	45 - 47	Grey varved silty clay.	450						P
	SS10	50 - 52	Grey silt.							P

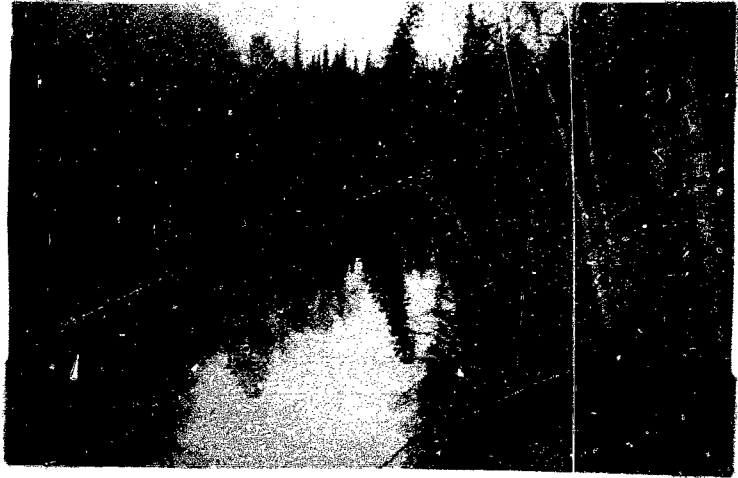
TABLE No. 3 (cont'd)

Hole No.	Sample No.	Depth Ft.	Description	Shear Strength		Consistency			Unit Wt. p.s.f.	N
				p. s. f. Vane*	C.	% dry wt. W	L.L.	P.L.		
3	SS1	5 - 7	Grey sandy stony silt.							
	SS2	10 - 12	Grey clayey silt.	290						P
	SS3	15 - 17	Grey varved silty clay.	280						P
	SS4	20 - 22	Grey varved silty clay.	350						P
	SS5	25 - 27	Grey varved silty clay.	480						P
	TW6	30 - 32	Grey varved silty clay.	420	520	67.7*			107.2	P
	SS7	35 - 37	Grey varved silty clay.	455		53.1				P
	SS8	40 - 42	Grey varved silty clay.	580						P
	SS9	45 - 47	Grey varved silty clay.	550						P
	SS10	50 - 52	Grey silt.							P
4	SS1	10 - 12	Grey med. to coarse grained silt.	305						P
	SS2	20 - 22	Grey varved silty clay.	470						P
	TW3	30 - 31½	Grey varved silty clay.	450	320	71.2*			105.2	P
	SS4	40 - 42	Grey varved silty clay.	525		52.6				P
	SS5	50 - 52	Grey coarse grained silt.							P
	SS6	60 - 62	Grey silt with odd clay layer.							P
	SS7	70 - 72	Grey silt with odd clay layer.							P
	SS8	80 - 82	Grey medium grained silt.							P
	SS9	90 - 92	Grey sandy coarse silt.							P
	SS10	100 - 102	Grey silt.							P

Notes: Vane* - vane strengths represent stratum cohesion at 1½ feet below the sample depth indicated.
 C - apparent cohesion in terms of total stresses.
 W - water content composite sample (% dry weight).
 W* - water content clay phase of varve (% dry weight).
 L.L. - liquid limit (% dry weight).
 P.L. - plastic limit (% dry weight).
 N - number of blows per 1 foot penetration of 2" O.D. split spoon with driving energy of 350 ft.lbs./blow (i.e. standard penetration test).
 P - sampler pushed into stratum.

TABLE No. 4

Little Gravel River
Upstream from Bridge
Site.



Bridge Approaches
Looking West.

Little Gravel River
Downstream from Bridge
Site.

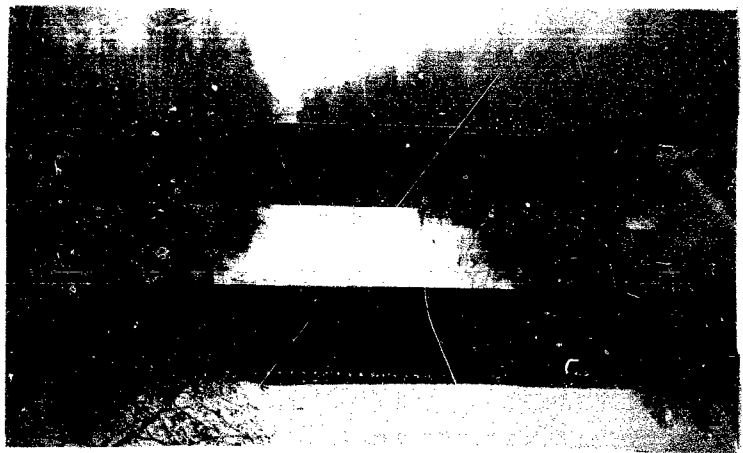


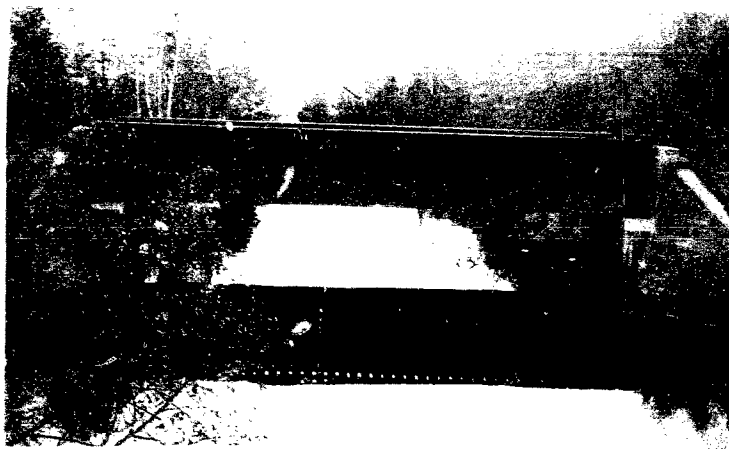
TABLE No. 4.

Little Gravel River
Upstream from Bridge
Site.

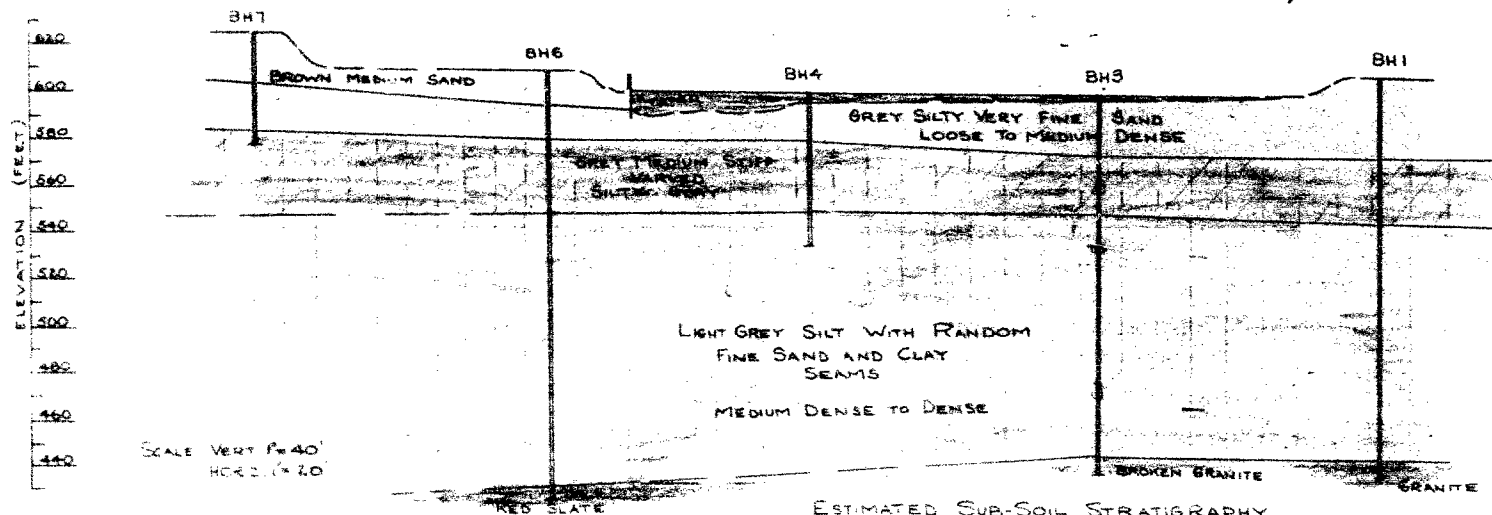
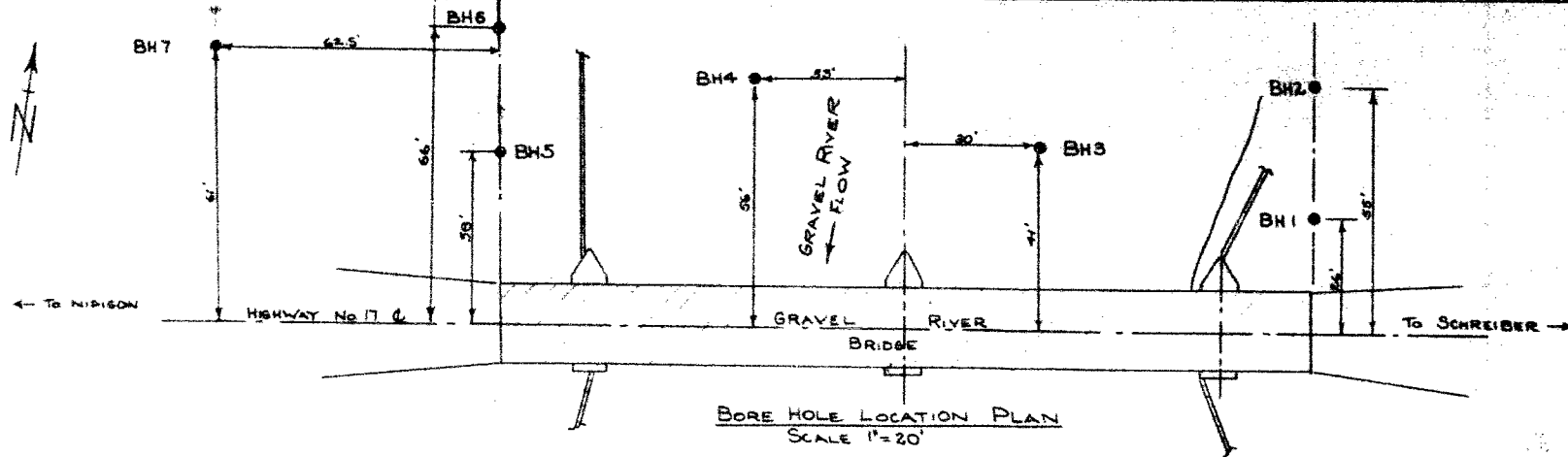


Bridge Approaches
Looking West.

Little Gravel River
Downstream from Bridge
Site.



DRAWING No. 1
JOB No. C108/J246B



SCALE VERT 1"=40'
HORIZ 1"=20'

GRAVEL RIVER

BORE HOLE LOCATION

AND

SUB-SOIL STRATIGRAPHY

SCALE: AS SHOWN

DWN. BY: K.P.

CHKD. BY: L.G.S.

OCT. 1958