#58-F-270-C

W.P.#914-58

Hwy. #17 T.C.H.

CROSSING

GRAVEL RIVER

BA 834

TROW, SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

W. A. TROW, M.A.SC., M.E.I.C., P.ENG.
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884 WILSON AVE.,
DOWNSVIEW, ONT.
ST. 8-5921

Project: J268

November 17, 1958

58-F-270C

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

Attention Mr. S. McCombie

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

Dear Sirs:

Submitted herewith is our report containing the results of a foundation investigation recently completed at the above site. In addition to a presentation of factual data obtained from the field and laboratory work carried out we have included for your consideration an evaluation of the competency of the subsoil types to support the bridge loadings. A summary of the principle comments arising out of this investigation is as follows:

1) The subsoil profile at this bridge consists of a) an upper stratum of fine grained granular material grading from medium sand to very silty fine sand. This deposit exists in a loose to medium dense state. b) A middle layer of medium stiff varved silty clay. This stratum is continuous across the site and varied in thickness from 26 to 33 feet. c) A lower deposit of medium dense to dense silt. This silt stratum was found to be greater than 100 feet in thickness. This deposit was found to overlay the bedrock which was proven at 3 borehole locations.

2) The variation in relative density of the upper granular deposit is such that spread footing founded in this stratum is not recommended.

3) Pile supported piers and abutments appear necessary. Bearing piles driven to refusal at the bedrock contact would provide positive footing support. In order to determine whether or not piles to bedrock are necessary a study of allowable pile capacity and settlement assuming a range of dead load reactions has been made. The results of the analysis are presented in tables contained in the body of this report. These results indicate that satisfactory footing support can be

obtained through using pile lengths of 50 feet or 70 feet. The choice is dependent upon the actual design loads which are not available at this time.

4) The condition of the existing river banks and the scour and fill action which is evidenced at the existing bridge location attest to the activity of the river with regard to scour, fill and bank erosion. A study of the effectiveness of the existing river bank and pier protection should be undertaken prior to designing protective means for the new structure.

5) The proposed grade line indicates that embankment heights will be well within safe limits with regard to foundation failure or overstressing.

We are pleased to have been of service to you on this occasion. If we can be of assistance in clarifying or substantiating data contained herein please do not hesitate to contact our office.

Yours very truly,

L. G. Soderman

Lawrence G. Soderman (P. Eng.)

LGS/kb
ENC.

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

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

Project: J268 November 17, 1958

Trow Soderman and Associates

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ENCLOSURES

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

Reported herein are the results of a sub-surface exploration program recently completed at the above noted site. This report contains the detailed results of field and laboratory findings, along with recommendations for the proposed foundation at this site.

Site Location and Description

Gravel River crosses the existing Highway No. 17 approximately 28 miles west of Schreiber, Ontario and 94 miles east of Port Arthur, Ontario. At present the river is crossed by a bridge consisting of two 60 ft. timber truss spans and two 15 ft. timber approach spans. The existing timber bridge is reinforced with a single lane Bailey structure placed inside the wooden trusses. This bridge is supported by two centre pile bents and two approach pile bents containing piles approximately 40 feet long. These pile bents are surrounded with protective vertical wooden planking.

The proposed Gravel River Bridge is located immediately north of the existing bridge. Both bridge sites are situated at the apex of a horse shoe river bend. At the point of crossing the river flows in a general south direction while immediately up and down stream the river flows west and east respectively. Because of the horse shoe shaped course of the river active erosion takes place along the westward shore, while deposition is evident on the easterly side of the river. Both sides of the river in the vicinity of the existing bridge are protected from erosion by wooden pile walls, in addition the down stream west shore is protected with rock fill beyond the wooden pile wall. The course of the river is irregular with several meander loops in the immediate vicinity of the bridge location. Gravel River flows into Lake Superior through a man made outlet ½ mile down stream from the bridge location. The present river outlet was constructed to eliminate several thousand feet of meanders and thus facilitate logging operations.

The terrain surrounding the bridge location, with the exception of a 15 to 20 foot ridge on the upstream west bank, is generally flat. The higher ridge which slopes gently to the west is gradually being eroded by the river. The adjacent area is covered with a medium growth of birch, spruce and poplar trees.

Field and Laboratory Work

The field work associated with this investigation was carried out during the period of September 24 to October 7th. During this period seven boreholes with adjacent dynamic cone penetration tests were driven at locations shown on drawing No. 1.

Borings were carried out using a standard diamond drill adapted for soil sampling. The holes were advanced by alternately driving and washing 3-1/16 inch or 2-15/32 inch I.D. casing. Boreholes No. 3 and 4 were placed in the river with the aid of a portable floating raft. Boreholes No. 1, 3 and 6 were advanced until bedrock was encountered and samples of the bedrock obtained. The remaining 4 bore holes were carried to a depth sufficient to determine all strata horizons and to obtain sufficient samples of each stratum.

In cohesive material undisturbed samples were obtained using a 2 inch I.D. thin walled Shelby tube sampler. In non-cohesive type soils a 2 inch O.D. split spoon sampler was used. The split spoon sampler was driven in accordance with the requirements of the standard penetration test, i.e. 350 ft.lbs per blow. The number of blows (N) required to drive the spoon one foot are recorded in the summary table 3. Where disturbed samples in the cohesive stratum were considered sufficient the split spoon sampler was used. All samples were sealed to prevent moisture changes, and transported to the laboratory for testing and further visual identification.

In addition to the cohesive strength values obtained from undisturbed samples in-situ vane tests were carried out in the cohesive material giving additional shear strength values. These values along with the sensitivity (i.e. ratio of undisturbed to remoulded strength) are indicated on the borehole logs, drawings 2 to 8.

As a supplement to the standard penetration tests carried out in the non-cohesive strata, dynamic cone penetration tests were performed adjacent to each borehole location. If the borehole was not carried to bedrock additional dynamic cone penetration tests were carried out at the bottom of the borehole. These tests in the bottom of the boreholes help to evaluate the effect of friction in the overlying material on the driving rods.

The borehole locations were obtained by measuring offsets from the centre line of the existing bridge. The offset was determined by scaling the distance on drawings supplied by the Ontario Department of Highways. The elevation of each borehole was determined by hand level and referenced to the water level indicator found at the west abutment.

Undrained triaxial shear tests were performed on the undisturbed samples, while moisture, density and Atterberg limit determinations were carried out on selected samples. In addition to these tests one consolidation test and one hydrometer test was performed.

Soil Types Encountered

A detailed description of the soil types encountered, along with the elevations of the upper and lower horizon of each stratum is given in the borehole logs, drawings 2 to 8. In addition to the detailed description of each bore hole a cross section of the estimated sub-soil stratigraphy is presented as drawing No. 1.

Brown Medium Sand. This material is found on the west shore of the river, from the ground surface to a depth of 10 to 20 feet. It exists in a loose to medium dense state, with the looser areas closer to the river bank. The brown sand is generally clean and contains a small percentage of gravel. Open pits in the vicinity of the bridge indicate that this material has been used as fill.

Grey Silty Very Fine Sand. This deposit of fine grained granular material was found to underlie the entire site. It varies in thickness from 30 feet in the vicinity of borehole 1, where it is found at the surface, to 20 feet at borehole 7 where it is overlain by the brown medium sand. Dynamic cone blows and standard penetration test results indicate that the material is loose to medium dense (i.e. N varies from 6 to 20). The stratum contains small amounts of gravel and occasional silt layers, but retains the characteristics of a fine to very fine sand.

Grey Varved Silty Clay. Underlying the fine sand a deposit of grey varved silty clay was encountered in each borehole. The silty clay was described as medium stiff with laboratory strength values of cohesion ranging from 300 to 1000 psf. In-situ vane strengths in this material varied from 500 to slightly above 1000 psf. A representative average value of shear strength is 750 psf.

The moisture content of composite samples of the clay (i.e. containing both silt and clay portions of the varve) gave values of near 45% of dry weight. The uniformity of the values from different depths and different locations is an indication of the homogenity of the material. Moisture contents were taken of the clay portion of a varve and gave higher values ranging to 60% dry weight. The lack of uniformity in the moisture content tests of the clay phase was due to the difficulty in cutting out the clay portions of the sample, as in some samples the varving was barely perceptable at natural moisture content.

Atterberg limit tests were carried out on two samples and gave liquid limits ranging from 71 to 55% while the plastic limits varied from 27 to 25%. The unit weight of the silty clay material was found to be approximately 110 psf. Exact values of all test results are summarized in table No. 3.

An odeometer test carried out on one selected relatively undisturbed sample taken from the lower portion of the stratum gave a compression index of 0.51. This value was taken directly from the uncorrected laboratory test curves of e vs. log p. The curve is considered characteristic of a normally loaded medium compressible deposit.

Light Grey Silt. Light grey silt was intersected immediately below the silty clay. The grey silt varied in thickness from 100 to 120 feet. The deposit is medium dense to dense as indicated by the dynamic cone penetration profiles. Very thin layers of fine sand and clay where encountered at random depths throughout the silt stratum. These layers vary from 1/16 to 1/8 of an inch in thickness and are sufficiently widespread to have little effect on the characteristics of the silt.

A hydrometer test on a representative sample of the grey silt (drawing No. 10) indicates that the silt consists mainly of particles in the .010 to .015 mm size, with some particles approaching the size of fine sand. For settlement calculations a value of 0.2 for the compression index (C_c) was assumed. Typical values of compression index for similar silts range from 0.15 to 0.2.

Bedrock. Bedrock was encountered at approximately 160 feet from the surface. Rock core recovered from borehole 1 was sound with good core recovery and is classified as a granite pegmatite. Bedrock in borehole 3 was identical in composition to borehole 1, but contained numerous fractures. Considerable flow of water under artesian head was encountered at the surface of the rock in borehole 3. Borehole No.6 revealed a reddish slate type bedrock entirely different from the previous holes, indicating a transition zone in the vicinity of the broken rock in borehole 3.

Foundation Considerations

Due to the deep deposit of overburden and the loose nature of the top strata large displacement piles (i.e. wood or monotube) are best suited to support the proposed bridge. These large displacement piles may be terminated in the clay stratum or driven into the underlying silt deposit. The length of pile selected will depend on the abutment and pier loadings. For convenience in selecting the proper pile length to correspond with pier loading calculations have been made to cover various loads.

The shear strength of the clay has been evaluated at 750 psf. This value is based on the results of in-situ field vane shear tests, and quick undrained triaxial tests carried out in the laboratory. In calculating the capacities of piles founded in this varved clay stratum a value of adhesion equal to 0.8 of this value (i.e. 600 psf) has been used. The reduction factor is in accordance with recent research published by M.J. Tomlinson (Fourth International Conference on Soil Mechanics 1957).

Settlements resulting from the pier and abutment loads have been calculated using the compression indexes (C_c) of 0.51 and 0.2 for the clay and silt respectively. The value of 0.51 was obtained from the consolidation test results (drawing No. 7) of a representative relatively undisturbed sample, while the value of 0.2 has been estimated and is considered typical for this type of silt.

The pile design may be classified into two groups. 1) 50 foot piles terminating near the bottom of the clay stratum and 2) 70 foot piles driven some 15 feet into the silt stratum. In both the above cases the piles will not meet with refusal but will carry their load by both skin friction and end bearing. In the case of the 50 foot piles skin friction carries the major portion of the load, while end bearing is the major consideration on the piles founded in the silt.

With a pile length of 50 feet the individual pile capacity for a 12 inch diameter pile has been evaluated as 8.76 tons (design sheet No. 1). The total capacity of a pile group containing 30 piles (i.e. abutment 30 ft x 6 ft containing 3 rows of 10 piles each) is thus 263 tons. This value of load capacity assumes that the pile tip elevation equals 550.0 ft thus utilizing a 25 foot depth of clay to provide skin friction and placing the top of pile below low water level. The individual capacities for pile diameters of 8 inches and 15 inches have been calculated and presented in table No. 1.

Pile Capacity Assuming 70 Foot Pile

Using a 70 foot pile with top below low water level the pile tip terminates in the silt material at approximately elevation 530.0. The pile capacity is dependent mainly on the end area of the pile and the density of the granular material. From Meyerhof A.S.C.E. January 1956 we find that the ultimate capacity of piles in granular material is $4 N A_p$ + skin friction,

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

A_p = sectional area of pile

and the answer is in tons. For the silt stratum encountered at this site and for 70 foot piles an average N value of 20 was selected from results of dynamic cone penetration tests. From design sheet No. 2 we find that the maximum load on such a pile 12 inches in diameter is 29.1 tons. Similar calculations for 8 and 10 inch diameter piles have been made and are presented in table No. 1.

Pile lengths of 50 and 70 feet have been given assuming that the top of pile will be just below low water level. Pile lengths may be adjusted accordingly provided the pile tips are at elevation 550.0 and 530.0 for the piles described as 50 feet and 70 feet respectively.

TABLE No. 1

Pile Length Feet	Pile Diameter Inches	Max. Pile Capacity Tons per Pile
50	8	5.6
50	10	7.2
50	12	8.8
70	8	14.9
70	10	21.5
70	12	29.1

Settlement calculations have been made for a series of dead loads acting on pile groups and presented in table No. 2. These values of settlement assume a 2 : 1 load distribution acting from the lower 1/3 point of the pile. Typical calculations for settlement of a 50 foot

12 inch diameter pile are presented on design sheet No. 3. Settlement figures stated for a 50 foot pile have been based on a 30 pile group placed in 3 rows of 10 piles each at 3 foot centres resulting in an approximate abutment or pier size 6 x 30 feet. The 70 foot pile settlements are assumed to have resulted from 2 rows of 10 piles each at 3 foot centres with the rows 4 feet apart, giving an approximate abutment size 30 x 4 feet.

TABLE No. 2

Pile Length Feet	Total Dead Load Tons	Settlement Inches
50	150	4.2
50	180	5.2
50	220	6.3
70	250	3.5
70	300	4.1

The time for completion of settlement resulting from dead loads as tabulated in table No. 2 is dependent upon the material supporting the load. In the case of the 50 foot pile founded in clay 90% of settlements will be complete in approximately 4½ years. Piles 70 feet long founded in silt should have completed settlement with the final application of dead load or the completion of the bridge structure.

In the design of the bridge foundations serious consideration should be given to the problems of river erosion. Protective measures provided for the existing bridge will provide some indication as to what will be required for the proposed bridge. River scour may present some problem to centre piers, thus these piers should be placed below any elevation known to be subject to scour. Scour on a piled foundation is not as critical as that of a spread type, but recommended procedure is to place the base of the foundation at a depth below river bottom equal to not less than four times the greatest known rise of the river level.

Summary

Piled foundation are recommended for this bridge. The pile length and diameter will depend on the abutment and pier loads. Knowing these loads the values may be obtained from the table No. 1 presented in the body of this report. Settlements associated with the typical design dead loads are considered to be within acceptable limits for this type of structure. The values are given in table No. 2.

Piles such as those recommended will not meet refusal but should be driven to the tip elevation recommended for the particular load required. It is advantageous to use the same pile length over the entire structure to ensure ease of pile driving and to minimize the differential settlement.

Attention should be given to possible river erosion at the abutments and river scour at the piers.

L. G. Soderman

for Kenneth R. Peaker (P. Eng.)

KP/kb

REGISTERED PROFESSIONAL ENGINEER
CIVIL
K. PEAKER

PROJECT NO. C108/J268

DRAWING NO. 2

TROW SODERMAN AND ASSOCIATES

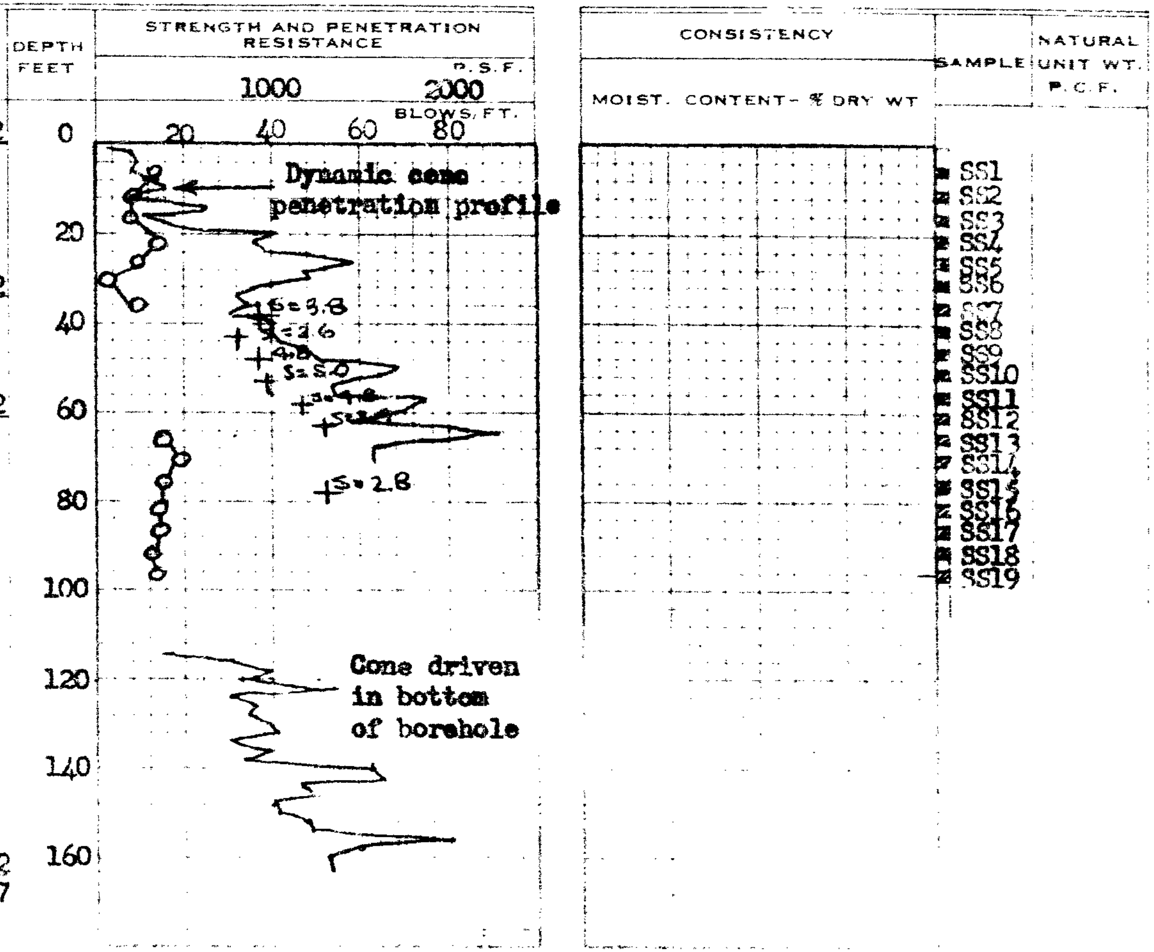
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Gravel River Bridge
LOCATION Highway No. 17
HOLE LOCATION See plan
HOLE ELEVATION AND DATUM 610.2

BOREHOLE NO. 1
FIELD SUPERVISOR D.S.
DRILLER E.S.
PREP. K.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

SYMBOL	DESCRIPTION	ELEV. FEET
	Ground surface.	610.2
	Grey loose to medium dense silty very fine sand.	575.2
	Grey medium stiff varved silty clay.	548.2
	Light grey silt with random thin fine sand and clay seams.	446.2
	End of hole.	438.7



Coarse sand and gravel 164 - 166 feet.
166 - 171½ recovered 90% NX core, reddish grey granite.

Ground water level 8.5 ft.

PROJECT NO. C108/J268

DRAWING NO. 3

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Gravel River Bridge
LOCATION Highway No. 17
HOLE LOCATION See Plan
HOLE ELEVATION AND DATUM 607.7

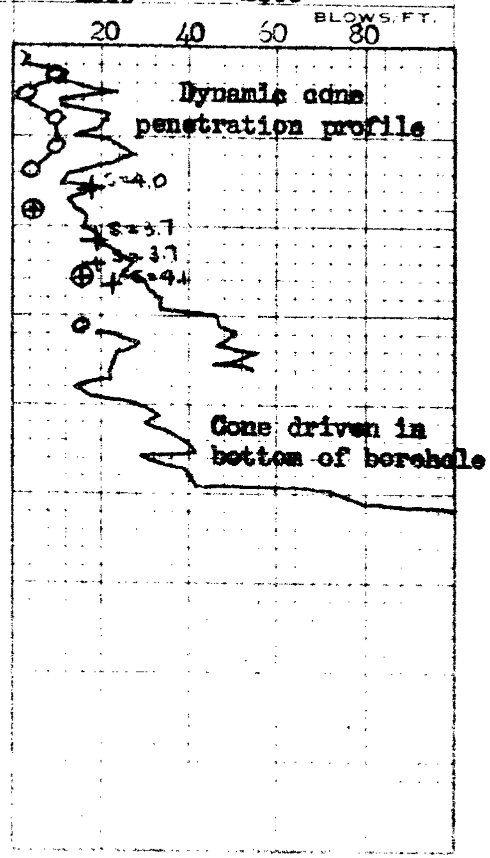
BOREHOLE NO. 2
FIELD SUPERVISOR K.P.
DRILLER E.S.
PREP. K.P.

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

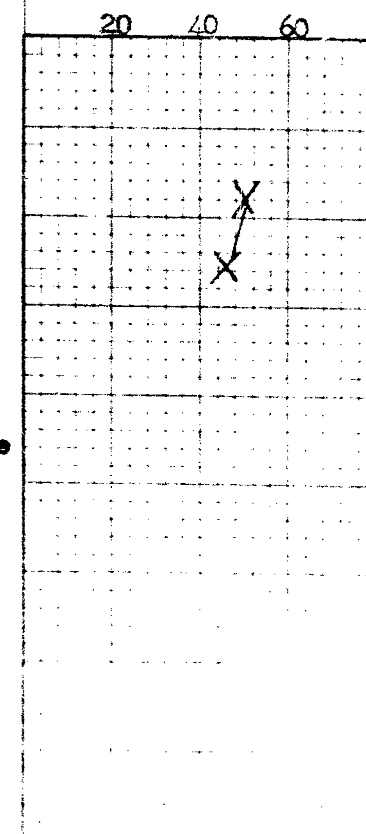
SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Ground surface.	607.7	0
	Grey fine to very fine sand with some silt.		
		577.7	
	Grey medium stiff varved silty clay.		
		549.7	
	Grey silt.	545.7	
	End of hole.		

Ground water level 6 ft.

STRENGTH AND PENETRATION RESISTANCE



CONSISTENCY



SAMPLE	NATURAL UNIT WT. P.C.F.
SS1	
SS2	
SS3	
SS4	
SS5	
SS6	
TW7	111.1
SS8	
SS9	
TW10	110.9
SS11	

PROJECT NO. C108/J268

DRAWING NO. 4

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

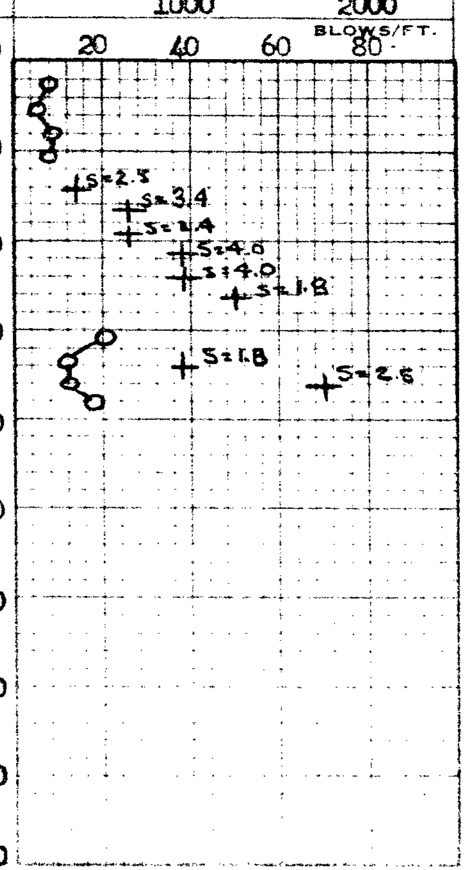
LEGEND
- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION (Qu)
- VANE TEST (C) AND SENSITIVITY (S)
- NATURAL MOISTURE AND LIQUIDITY INDEX
- LIQUID LIMIT
- PLASTIC LIMIT

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

BOREHOLE NO. 3
FIELD SUPERVISOR D.S.
DRILLER H.J.
PREP. K.P.

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	River surface. River depth=2'	601.3	0
	Grey loose to medium dense silty very fine to medium sand.	599.3	
		575.3	20
	Grey medium stiff varged silty clay.		40
		549.3	
	Light grey silt with random fine sand and clay seams.		60, 80, 100, 120, 140
		445.3	
	Red and grey granite.	437.3	160
	End of hole.		180

STRENGTH AND PENETRATION RESISTANCE



CONSISTENCY

MOIST. CONTENT - % DRY WT.

SAMPLE: SS1, SS2, SS3, SS4, SS5, SS6, SS7, SS8, SS9, SS10, SS11, SS12, SS13, SS14, SS15

NATURAL UNIT WT. P.C.F.

Cored AX 156'-164', recovery 100%.
Artesian flow 16 gpm at 8 foot head encountered at 165'.

PROJECT NO. 0108/ J268

DRAWING NO. 5

TROW SODERMAN AND ASSOCIATES

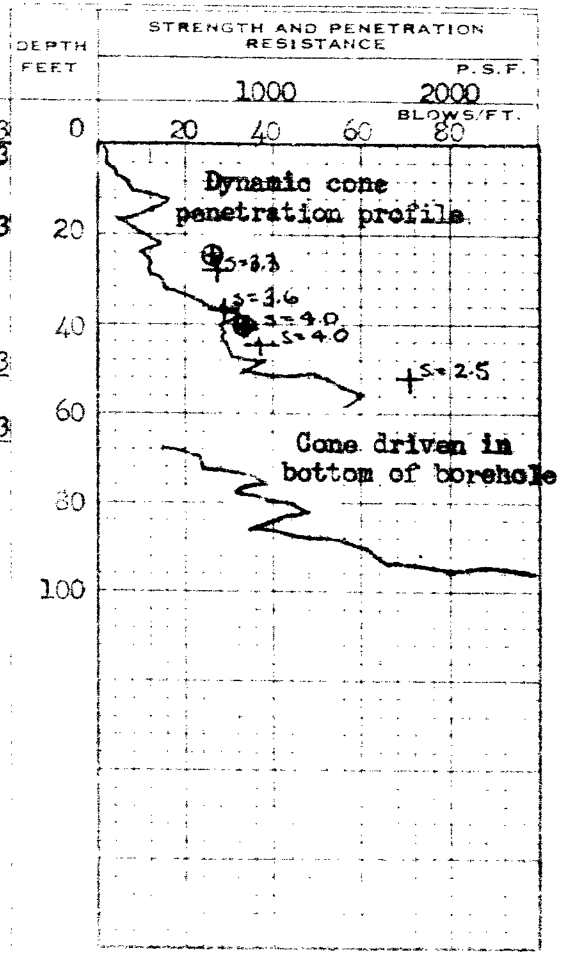
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

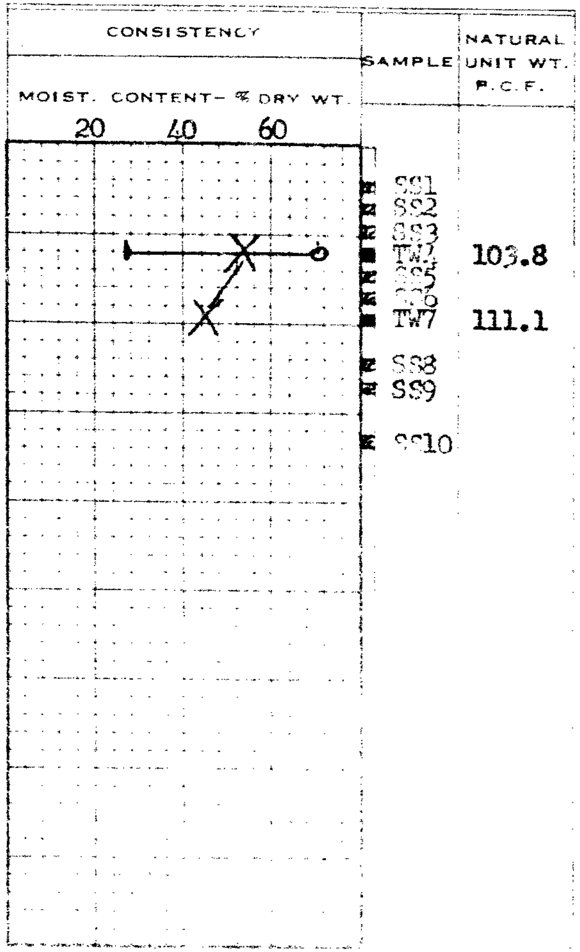
LEGEND
- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION (Qu)
- VANE TEST (C) AND SENSITIVITY (S)
- NATURAL MOISTURE AND LIQUIDITY INDEX
- LIQUID LIMIT
- PLASTIC LIMIT

PROJECT Gravel River Bridge
LOCATION Highway No. 17
HOLE LOCATION See plan
HOLE ELEVATION AND DATUM 601.3

BOREHOLE NO. 4
FIELD SUPERVISOR K.P.
DRILLER H.J.
PREP. K.P.

SYMBOL	DESCRIPTION	ELEV. FEET
	River surface. River depth=4'	601.3
		597.3
	Grey loose medium dense silty very fine sand.	581.3
	Grey medium stiff varved silty clay.	549.3
	Grey silt with random layers of clay and fine sand.	535.3
	End of hole.	





PROJECT NO. C108/J268

DRAWING NO. 6

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

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

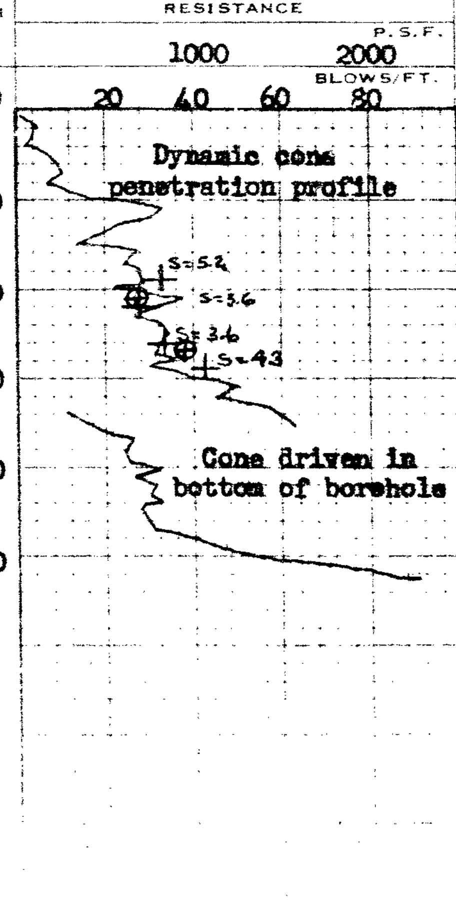
PROJECT Gravel River Bridge
LOCATION Highway No. 17
HOLE LOCATION See plan
HOLE ELEVATION AND DATUM 609.8

BOREHOLE NO. 5
FIELD SUPERVISOR K.P.
DRILLER H.J.
PREP. K.P.

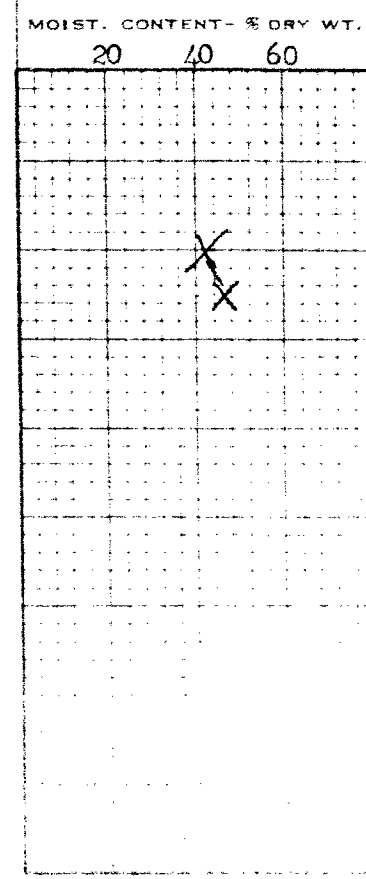
SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Ground surface.	609.8	0
	Brown changing to grey loose silty fine sand.	594.8	
	Grey fine sandy coarse silt.	576.8	20
	Grey soft to medium stiff varved silty clay.	549.8	40
	Light grey silt with random thin clay seams.	542.8	60
	End of hole.		

Ground water level = 3 ft.

STRENGTH AND PENETRATION RESISTANCE



CONSISTENCY



SAMPLE	NATURAL UNIT WT. P.C.F.
SS1	
SS2	
SS3	
SS4	
SS5	
SS6	
SS7	
TW8	108.1
TW9	109.5
SS10	
SS11	

PROJECT NO C108/J268

DRAWING NO. 7

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Gravel River Bridge

LOCATION Highway No. 17

HOLE LOCATION See plan

HOLE ELEVATION AND DATUM 610.7

BOREHOLE NO. 6

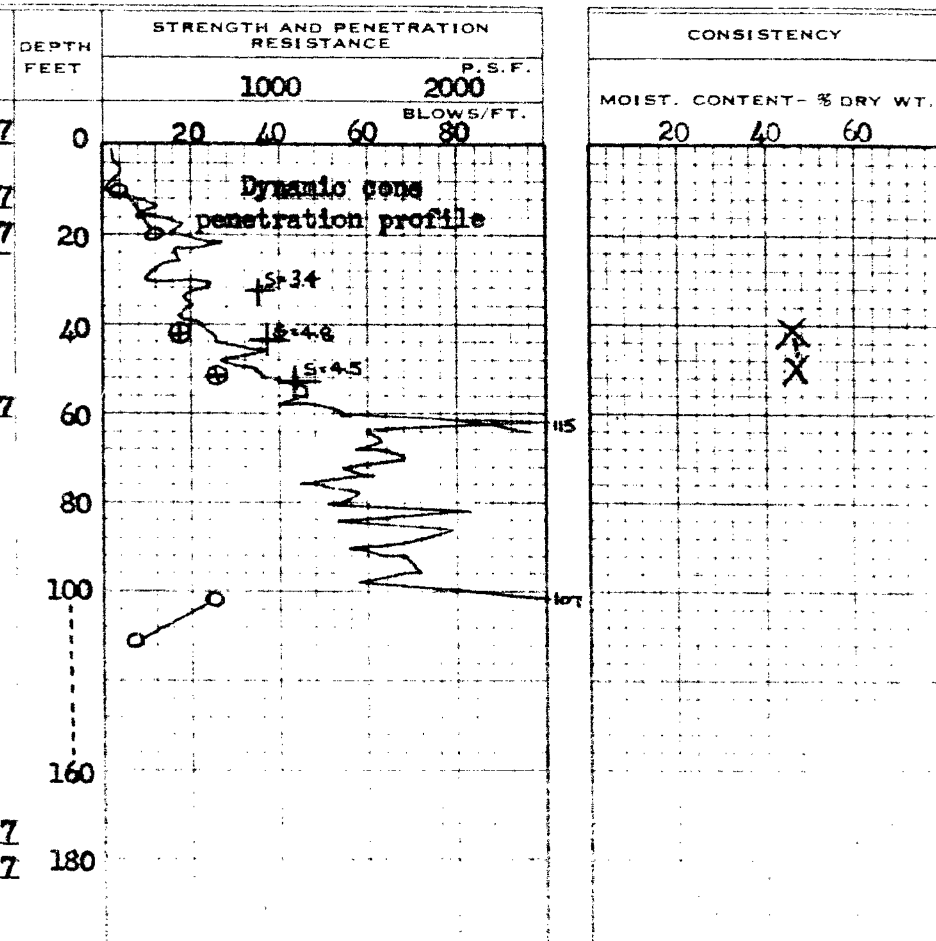
FIELD SUPERVISOR K.P.

DRILLER E.S.

PREP. K.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

DESCRIPTION	ELEV. FEET
Ground surface.	610.7
Brown and grey silty fine sand with some gravel.	595.7
Grey fine sandy silt.	581.7
Grey soft to medium stiff carved silty clay.	548.7
Light grey silt with random thin fine sand and clay seams.	
Wash down AX casing 110 ft. to refusal at 178 ft.	432.7
Soft red shale or weathered slate.	
End of hole.	426.7



SAMPLE	NATURAL UNIT WT. P.C.F.
SS1	
SS2	
SS3	
TW4	109.6
TW5	109.2
SS6	
SS7	
SS8	
SS9	
SS10	
SS11	

Core AX 178 to 184', recovery 10%.

Ground water level = 8 ft.

PROJECT NO C108/J268

DRAWING NO. 8

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

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

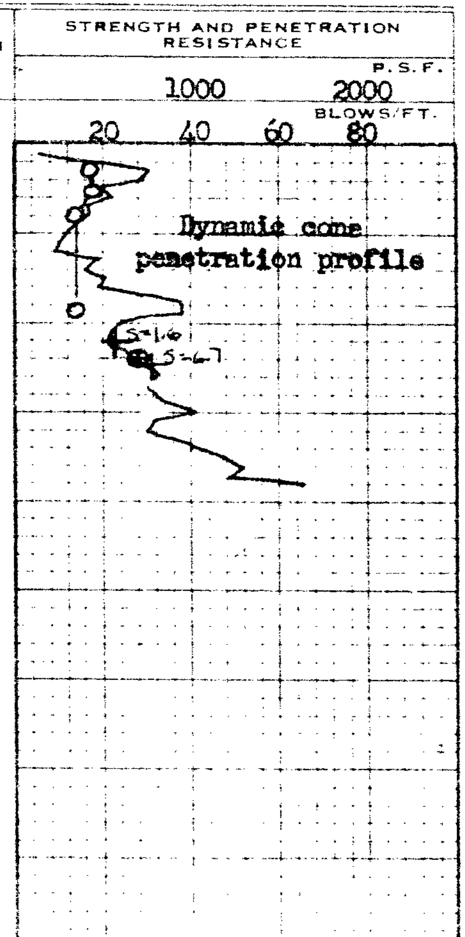
PROJECT Gravel River Bridge
LOCATION Highway No. 17
HOLE LOCATION See plan
HOLE ELEVATION AND DATUM 625.7

BOREHOLE NO. 7
FIELD SUPERVISOR K.P.
DRILLER H.J.
PREP. K.P.

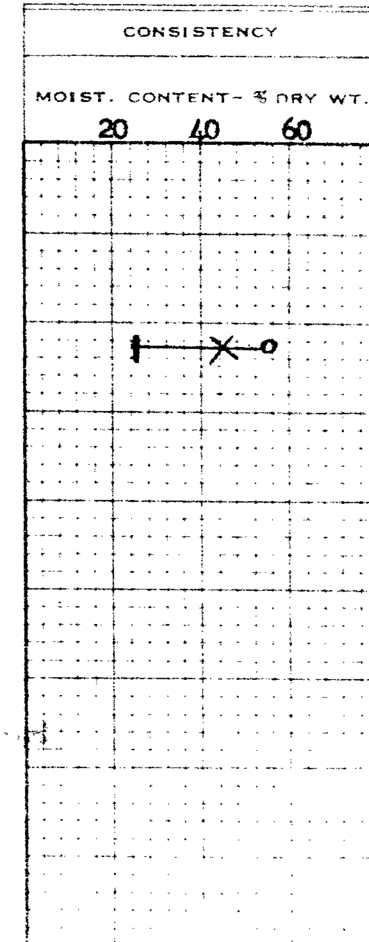
SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Ground surface.	625.7	0
	Brown with some grey medium dense medium to coarse sand.	604.5	20
	Grey silty fine sand.	584.7	40
	Grey varved med. stiff silty clay.	577.7	
	End of hole.		

Ground water level 18.5 ft.

STRENGTH AND PENETRATION RESISTANCE



CONSISTENCY



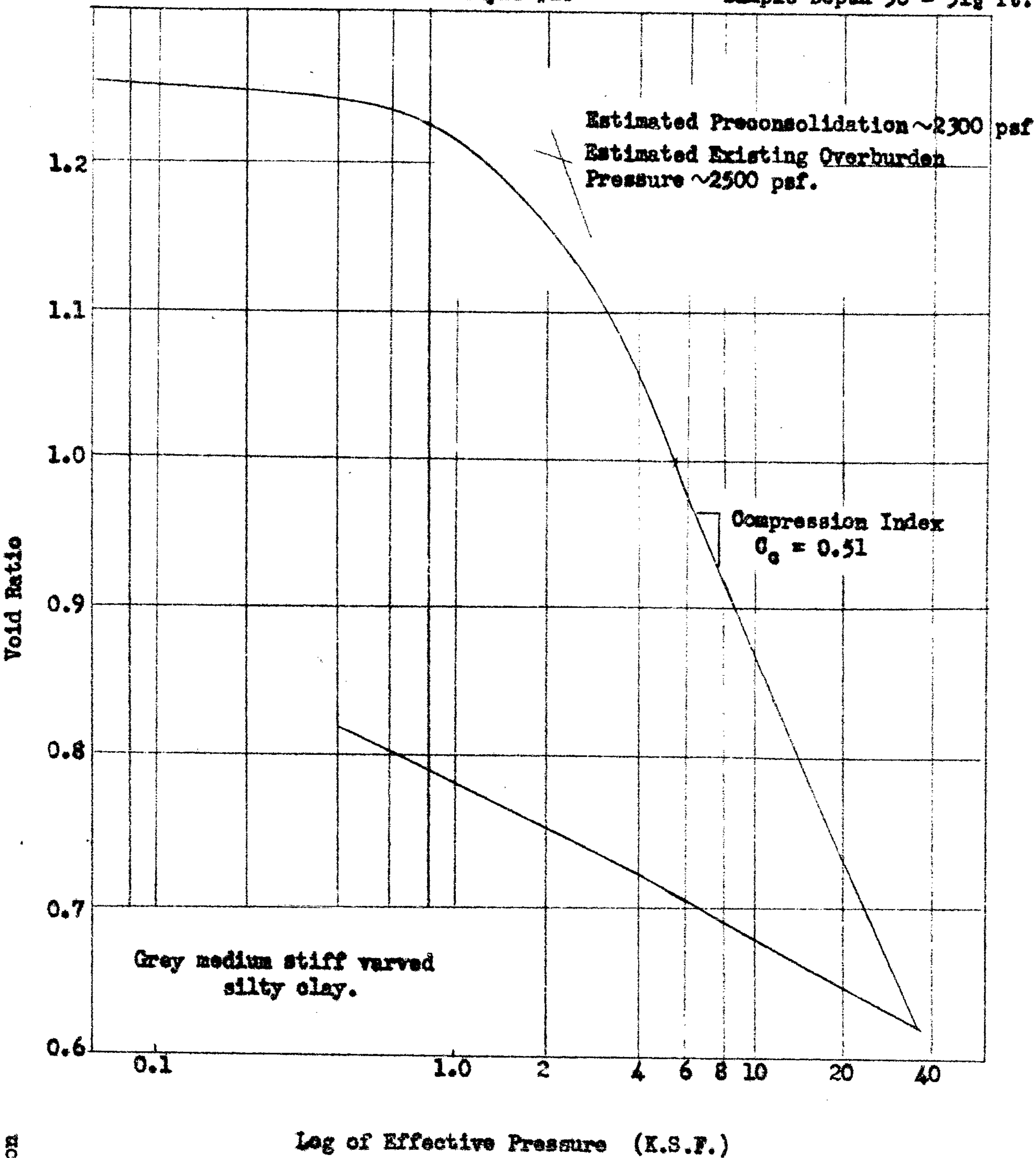
SAMPLE	NATURAL UNIT WT. P.C.F.
SS1	
SS2	
SS3	
SS4	
SS5	
SS6	
SS7	
SS8	
TW9	110.1

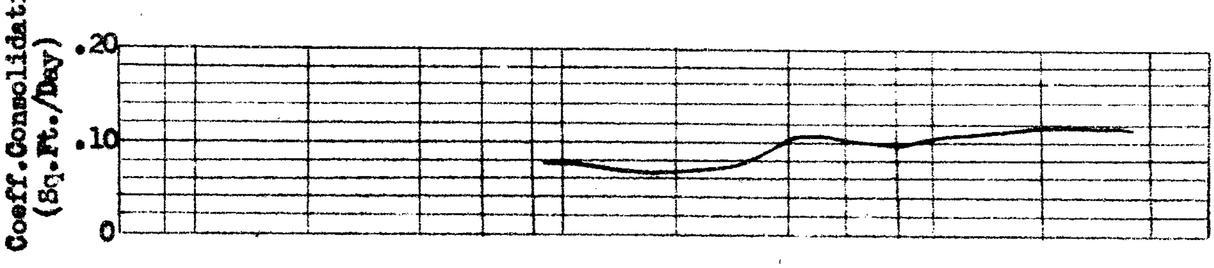
PROJECT NO. J268

DRAWING NO. 9

Uncorrected Laboratory Pressure Void Ratio Curve

BH #2 Sample #10 Sample Depth 50 - 51½ ft.



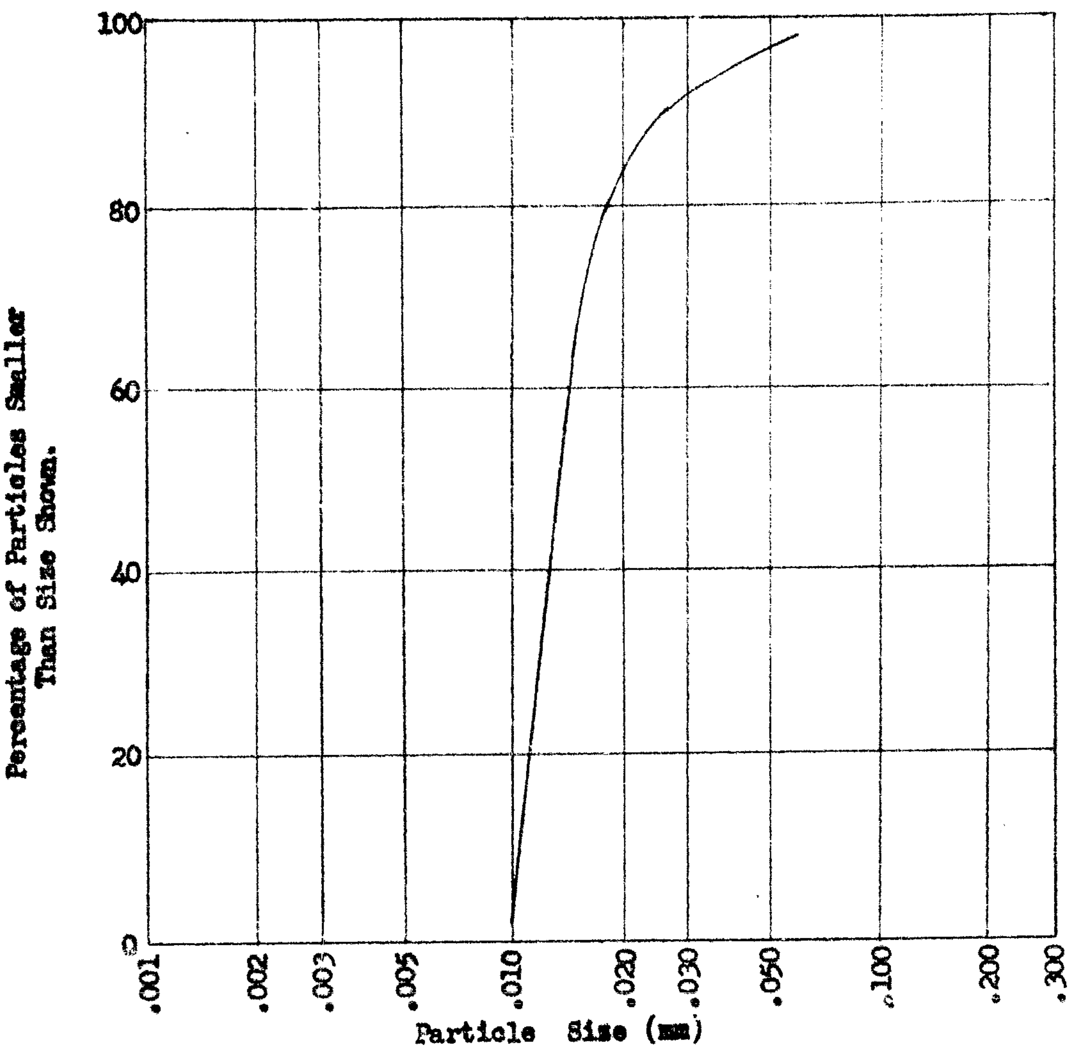


TROW SODERMAN & ASSOCIATES

MECHANICAL ANALYSIS

Hydrometer Test

BH #3 Sample SS13 Depth 65 - 67 ft.



CLAY	Fine	SILT	Coarse	SAND

M.T.I. Grain Size Classification

Design Sheet No. 1

Pile Length = 50 Ft. **Pile Diameter = 12 inches**

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

where A_s = area of pile surface subject to skin friction i.e. for 25 foot depth of clay and 12 inch pile 25×3.14 sq.ft.

C_c = cohesive strength corrected for adhesion = 600 psf.

C = cohesive strength = 750 psf.

N_c = bearing capacity factor = 9.

S.F.= safety factor of 3.

$$\text{Thus pile capacity} = \frac{25 \times 3.14 \times 600}{3 \times 2000} + \frac{750 \times 9}{3 \times 2000} = 8.76 \text{ tons.}$$

Capacity of 30 pile group 30 ft x 6 ft.

$= 30 \times 8.76 = \underline{263 \text{ tons}}$

This value of 263 tons is less than that calculated for the pile group acting as a deep foundation and therefore governs the design.

Design Sheet No. 2

Pile Length = 70 Ft. **Pile Diameter = 12 inches**

For pile capacity
load carried by pile tip $= \dfrac{4 \times N \times A}{S.F.}$

where N = standard penetration resistance = 20

A = sectional area of pile.

Load carried by skin friction $= \dfrac{A_s \times C_c}{F.S.}$

as in design sheet No. 1 considering 25 feet of clay stratum.

Thus total capacity of pile $= \dfrac{4 \times N \times A}{S.F.} + \dfrac{A_s \times C_c}{F.S.}$

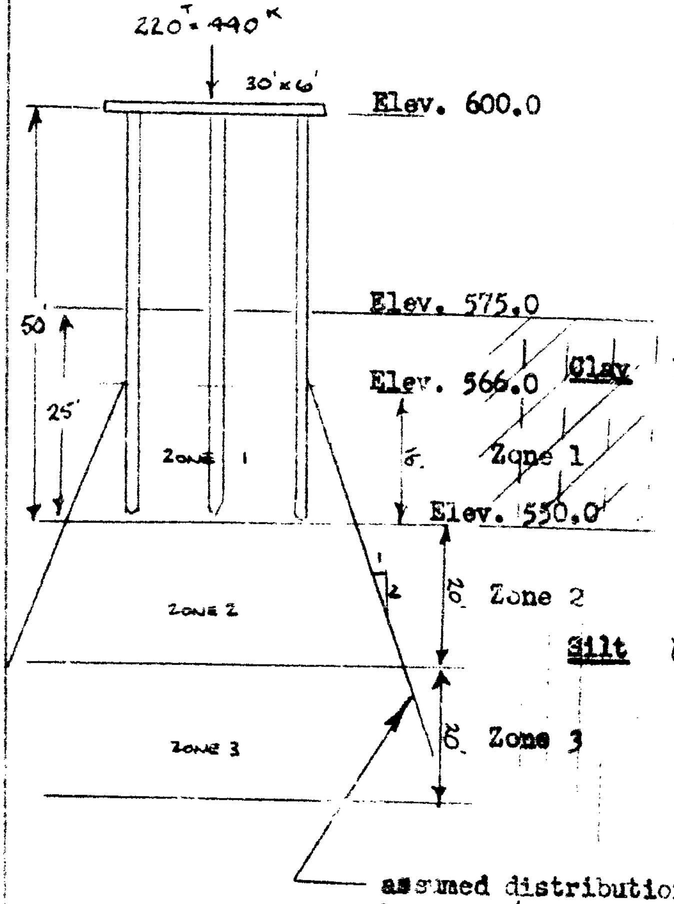
$$= \frac{4 \times 20 \times .8}{3} + \frac{25 \times 3.14 \times 600}{3 \times 2000} = \underline{29.1 \text{ tons}}$$

For pile group 4 x 30 feet containing 20 piles and carrying a dead load of 250 tons. Settlements calculated as in design sheet No. 3 give a value of total settlement

= 3.5 inches

Design Sheet No. 3

Settlement - 50 Ft. Pile
Assuming Total Dead Load of 220 Tons



γ submerged = 50 psf

Clay γ submerged = 50 psf

C_c = 0.51

e_o = 1.26

C = 650 C_o = 500

Silt γ submerged = 50 psf

C_c = 0.20

e_o = .95

assumed distribution of pressure at slope 2 : 1
bottom 1/3 of piles considered subject to settlement

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

where

S = settlement

H = depth of layer considered

C_c = compression index

e_o = initial void ratio

P_o = original overburden pressure

Δp = change in pressure.

Design Sheet No. 3A

Computations for Values of p_o

Assuming original ground level before river erosion = 614.0 and water table = 604.0:

$$\text{wt. of soil above water table} = \frac{10 \times 110}{1000} = 1.1^k$$

$$\text{wt. of soil to centre zone 1} = \frac{50 \times 46}{1000} = 2.3$$

$$\text{total} = 3.4^k = p_o$$

similarly p_o zone 2 = 4.3
zone 3 = 5.3

Computations for values of Δp

At centre of zone 1

$$\text{area loaded} = [6 + (2 \times 4)]\,[30 + (2 \times 4)] = 530 \text{ sq.ft.}$$

$$\therefore \Delta p = \frac{440}{530} = .831^k/\text{ sq.ft.}$$

similarly zone 2 = $.25^k$/ sq.ft.
zone 3 = $.113^k$/ sq.ft.

Δp at lower elevations may be considered negligible.

Settlement Computations

$$\text{zone 1} \quad S = \frac{16 \times 12 \times .51}{1 + 1.26} \log \frac{3.4 + .831}{2.1} = 4.3''$$

similarly zone 2 = .6"
zone 3 = .3"

Total	5.2 inches
+ 20% for elastic settlement	1.1
Total Settlement =	6.3 inches

TABLE No. 3

SUMMARY OF LABORATORY TEST RESULTS

Hole No.	Sample No.	Depth Ft.	Description	Shear Strength p. s. f. Vane*	C.	Consistency % dry wt. W+	L.L.	P.L.	Unit Wt. p.s.f.	N
1	SS1	5 - 7	Grey fine sand.							12
	SS2	10 - 11½	Grey silty fine sand.							5
	SS3	15 - 17	Grey silty very fine sand.							3
	SS4	20 - 22	Grey silty very fine sand.							14
	SS5	25 - 27	Grey silty very fine sand.							9
	SS6	30 - 32	Grey coarse silt.							1
	SS7	35 - 36½	Grey varved silty clay.	935						9
	SS8	40 - 42	Grey varved silty clay.	800						P
	SS9	45 - 46½	Grey varved silty clay.	925						P
	SS10	50 - 52	Grey varved silty clay.	975						P
	SS11	55 - 57	Grey varved silty clay.	1180						P
	SS12	60 - 62	Grey varved silty clay.	1300						P
	SS13	65 - 67	Grey coarse silt.							8
	SS14	70 - 71½	Grey coarse silt.							19
	SS15	75 - 76½	Grey coarse silt.	1300						15
	SS16	80 - 82	Grey coarse silt.							14
	SS17	85 - 87	Grey coarse silt.							14
	SS18	90 - 92	Grey fine to coarse silt.							12
	SS19	95 - 97	Grey fine sand.							13
2	SS1	5 - 6½	Grey fine to medium sand.							5
	SS2	10 - 12	Grey silty fine sand.							2
	SS3	15 - 17	Grey fine sand.							9
	SS4	20 - 22	Grey fine sand.							10
	SS5	25 - 27	Grey fine to coarse silt.							P
	SS6	30 - 32	Grey silty varved clay.							4
	TW7	35 - 36½	Grey varved silty clay.	910	625	49.6			111.1	P
	SS8	40 - 42	Grey varved silty clay.	970						P
	SS9	45 - 46½	Grey varved silty clay.	970						P
	TW10	50 - 51½	Grey varved silty clay.	1050	900	47.5+ 45.4			110.9	P
	SS11	60 - 62	Grey fine to coarse silt.							15

TROW, SODERMAN AND ASSOCIATES

TABLE No. 3 (cont'd)

Hole No.	Sample No.	Depth Ft.	Description	Shear Strength p. s. f. Vane*	C.	Consistency % dry wt. W+	L.L.	P.L.	Unit Wt. p.s.f.	N
3	SS1	5 - 6½	Grey sand and gravel.							7
	SS2	10 - 11½	No recovery							5
	SS3	15½ - 17	Grey silty fine sand.							9
	SS4	20 - 22	Grey silty very fine sand.							6
	SS5	25 - 27	Grey varved silty clay.	360						P
	SS6	30 - 31½	Grey varved silty clay.	630						P
	SS7	35 - 37	Grey varved silty clay.	630						P
	SS8	40 - 42	Grey varved silty clay.	945						P
	SS9	45 - 47	Grey varved silty clay.	975						P
	SS10	50 - 52	Grey varved silty clay.	1250						P
	SS11	55 - 57	Grey coarse silt.							P
	SS12	60½ - 62	Grey clayey very fine sand.							21
	SS13	65½ - 67	Grey fine to coarse silt.	950						11
	SS14	70 - 71½	Grey fine to coarse silt.	1760						12
	SS15	75½ - 77	Grey fine to coarse silt.							17
4	SS1	10 - 12	Grey gravel and sand.							6
	SS2	15 - 16½	Grey silty fine sand.							9
	SS3	20 - 21½	Grey silty fine sand.							3
	TW4	25 - 26½	Grey varved silty clay.	675	650	55.1	71.1	27.0	103.8	P
	SS5	30 - 31½	Grey varved silty clay.	725						P
	SS6	35 - 36½	Grey varved silty clay.	830						P
	TW7	40 - 41½	Grey varved silty clay.	900	850	61.5+ 45.4			111.1	PP
	SS8	50 - 51½	Grey varved silty clay.	1780						P
	SS9	55 - 56½	Grey coarse to med.grained silt.							P
	SS10	65 - 66½	Grey varved clayey silt.							2
5	SS1	5 - 6½	Grey & brown fine to med.sand.							P
	SS2	10 - 12	Grey silty fine sand.							P
	SS3	15 - 16½	Grey silty very fine sand.							P
	SS4	20 - 22	Grey fine sandy coarse silt.							P
	SS5	25 - 27	Grey silty very fine sand.							P
	SS6	30 - 32	Grey varved clayey silt.							P
	SS7	35 - 37	Grey varved silty clay.	810						P
	TW8	40 - 41½	Grey varved silty clay.	670	680	43.8			108.1	P
	TW9	50 - 51½	Grey varved silty clay.	820	980	47.4			109.5	P
	SS10	55 - 57	Grey varved silty clay.	1060						P
	SS11	65 - 67	Grey coarse silt.							P

TABLE No. 3 (cont'd)

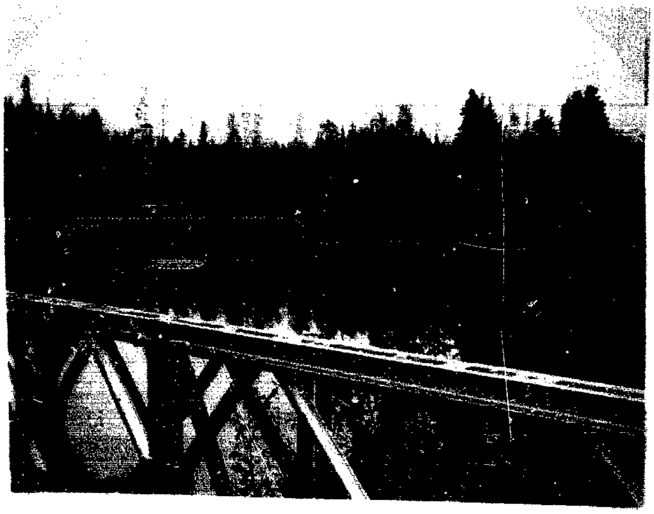
Hole No.	Sample No.	Depth Ft.	Description	Shear Strength p. s. f. Vane*	C.	Consistency % dry wt. W^+	L.L.	P.L.	Unit Wt. p.s.f.	N
6	SS1	10 - 12	Grey fine sand and gravel.							
	SS2	20 - 22	Grey fine sand and gravel.							P
	SS3	30 - 32	Grey varved silty clay.	880						11
	TW4	40 - 41½	Grey varved silty clay.	925	440	54.2^+				P
	TW5	50 - 51½	Grey varved silty clay.	1090	630	45.8			109.6	P
	SS6	60 - 61½	Grey clayey silt.			46.9			109.2	P
	SS7	70 - 71½	Grey fine to coarse silt.							P
	SS8	80 - 81½	Grey silt with some clay.							16
	SS9	90 - 92	Grey fine to coarse silt.	1020						P
	SS10	100 -101½	Grey medium silt.							P
	SS11	110 -111½	Grey coarse silt.							25
7	SS1	5 - 7	Brown clean coarse sand.							6
	SS2	10 - 12	Brown clean med. to coarse sand.							17
	SS3	15 - 17	Grey and brown clean coarse sand.							18
	SS4	20 - 22	as above changing to grey fine sand.							13
	SS5	25 - 27	Grey silty fine sand.							P
	SS6	30 - 32	Grey fine sand and coarse silt.							P
	SS7	35 - 37	Grey silty fine sand.							P
	SS8	40 - 42	as above changing to silty clay.	525						13
	TW9	45 - 46½	Grey varved silty clay.	770		45.9	54.5	24.6	110.1	P

<u>Notes:</u>

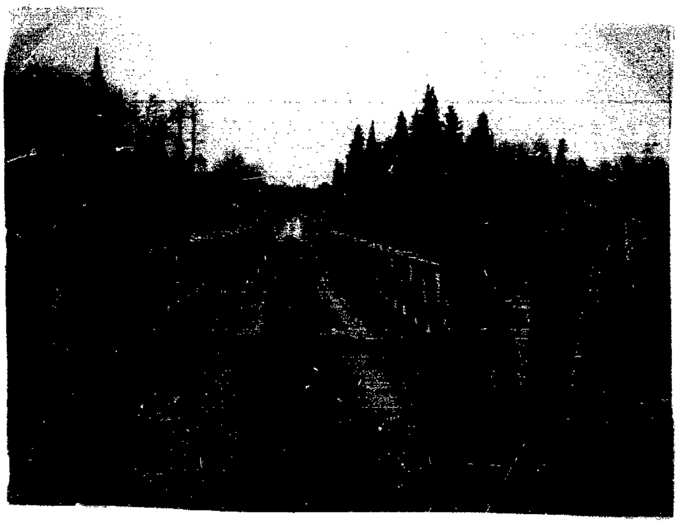
- Vane* - vane strengths represent stratum cohesion at 1½ feet below the sample depth indicated.
- C - apparent cohesion in terms of total stresses.
- W - water content composite sample (% dry weight).
- W^+ - water content clay phase of varve (% dry weight).
- L.L. - liquid limit (% dry weight).
- P.L. - 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

Existing Bridge Showing West Shore Erosion Protection.



Downstream View
Showing River Deposition

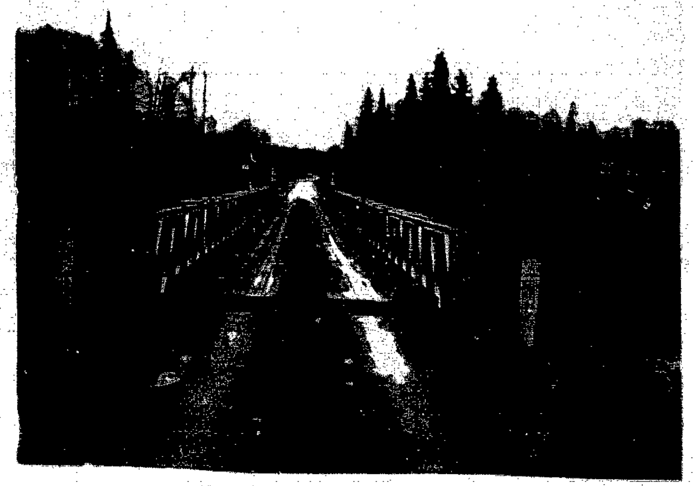


Bridge Approaches
Looking West

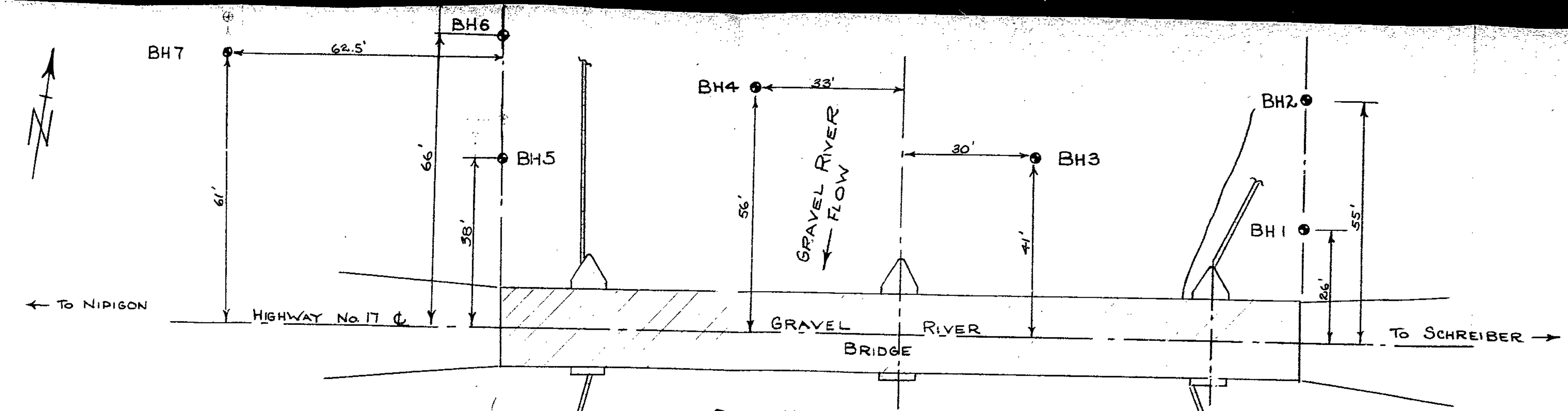
TABLE No. 4

Existing Bridge Showing West Shore Erosion Protection.

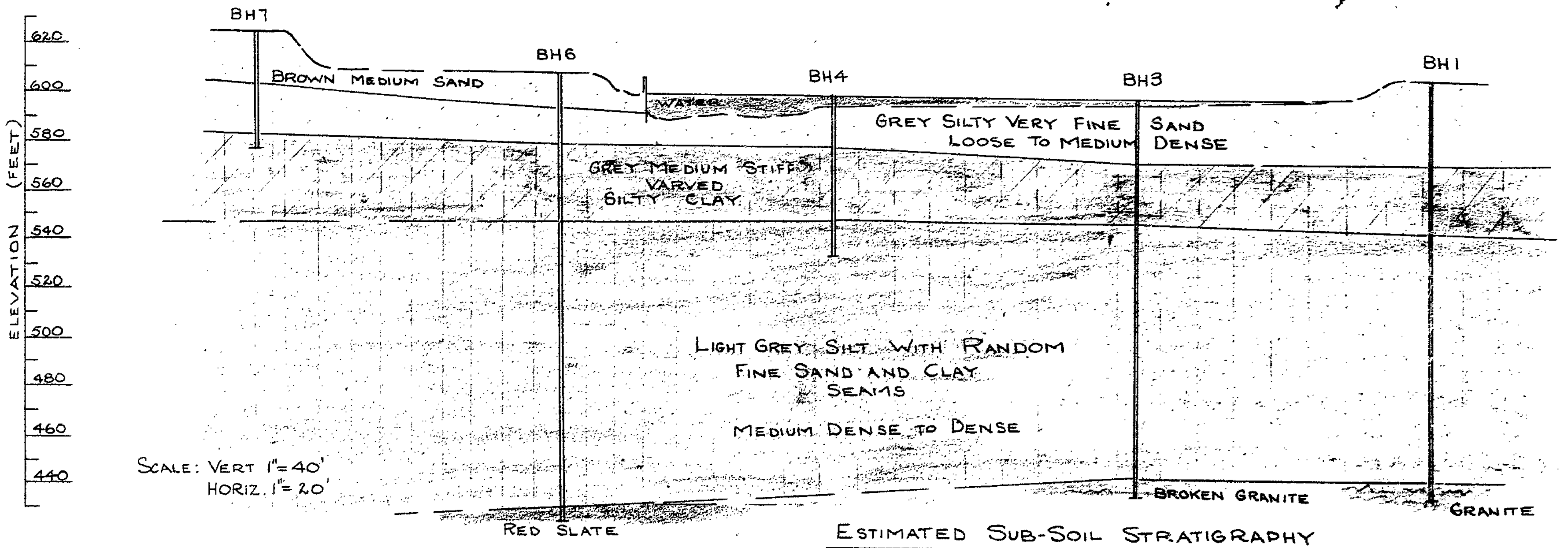
Downstream View
Showing River Deposition



Bridge Approaches
Looking West



Bore Hole Location Plan
Scale 1" = 20'



Estimated Sub-Soil Stratigraphy

Drawing No. 1
Job. No. C108/J268

GRAVEL RIVER

Bore Hole Location
And
Sub-Soil Stratigraphy

Scale: As Shown

Dwn. By: K.P.
Chkd. By: L.G.S.

Oct. 1958

c.c. Mr. S. McCombie.

FS
D19

BA 834

Mr. H.D. McMillan.
Asst. Road Design Engineer.
Materials & Research.

January 6th, 1959.

Re: Soils Report for Gravel River Crossing. W.P.91458. Hwy #17. T.C.H.

Attached find a soils design report containing a brief discussion of Trow, Soderman's foundation report, dated November 17th, 1958, and other pertinent data.

A spread footing type of foundation is not recommended due to the loose nature of the top strata. Instead, large displacement piles terminated in the clay stratum or driven into the underlying silt deposit will be necessary. Depending on loading considerations, piles of from 50 to 70 feet in length will have to be used. The 50' piles would terminate in the clay and therefore would depend mostly on skin friction to carry the load, whereas 70' piles would end in the silt and therefore end bearing would carry the major portion of the load. Therefore, it can be deduced, as pointed out in the foundation report, that 70' piles would carry higher loads and would result in complete settlement occurring with the final completion of the structure.

Serious consideration should be given to the problem of scour, which now exists as evidenced by the protection which has been necessary at the present structure and on the west shore downstream.

No soils profile is being submitted with this report. However, composite soils profile 17L83 (for W.P. 950-58) which includes this river crossing, will be available sometime in March.

A. RUTKA.
A/Mat'ls & Research Engr.

Per: T.J. Kovich

T.J. KOVICH.
Supervising Soils Engineer.

c.c. Mr. H.A. Tregaskes.
Mr. R.A. Panter.
Mr. J.B. Garland.
Mr. H.W. Hurrell.
Mr. H.A. Mantle.
Mr. S. McCombie.
Mr. A. Gray.
Mr. W. Bidell.
Mr. F. Gill.
Mr. A.C. Powell.
Mr. T.J. Kovich.
File.

tjk/zw.

SOILS DESIGN REPORT

GRAVEL RIVER CROSSING
HWY #17. (TCH).
W.P. 914-58

GENERAL DATA

The proposed crossing of the Gravel River by the Trans Canada Highway is approximately 28 miles west of Schreiber, Ontario, and 30 miles east of the intersection of Highways #11 and #17 at Nipigon. The site of the proposed structure is immediately north of the existing timber bridge and lies within the limits of grading W.P. 950-58 (Sta.1463±).

The Gravel River is quite wide and swift-flowing at the proposed site and meanders considerably. Erosion and deposition are quite severe on curves and some man-made control has been placed on the rate of erosion at the present site. Both the present structure and the proposed site are on the same horse-shoe bend, which is protected to some extent on the west side (the outside of the curve) by some 150 feet of continuous wooden piling.

The present structure consists of two timber trusses with a pier in the centre of the river. This bridge was reinforced two years ago by the addition of a single lane Bailey Bridge. The river has been used for navigation or logging operations in the past.

SOILS DATA & FOUNDATION CONSIDERATIONS

A foundation investigation was carried out by Trow, Soderman & Associates between September 24th and October 7th, 1958, with their report being issued on November 17th, 1958. For detailed results of field and laboratory findings, along with detailed foundation considerations, the Consultant's report should be referred to.

contd. p..2.

The sub-soil stratigraphy is described as follows:

1. On the west shore, from the ground surface to a depth of 10' to 20', is located a loose to medium dense layer of medium sand. This is fill material.
2. Underlying the sand fill a deposit of loose to medium grey silty very fine sand underlies the entire site at a thickness of from 20' to 30'.
3. Below the very fine sand is a stratum of medium stiff varved silty clay, varying in thickness from 26' to 33'.
4. Underlying the varved clay is a 100' to 120' thick layer of medium dense to dense light grey silt.
5. Bedrock was encountered at approximately 160' from the surface. The rock was granite at the east side, then changed to red slate on the west bank with the transition zone being somewhere under the river.

Due to the deep deposit of overburden and the loose nature of the top sand strata displacement piles are recommended. These piles can be terminated near the bottom of the clay stratum where their length will be 50' and they will depend mainly on skin friction for bearing capacity. An alternative pile length is 70', which would terminate the pile in the silt stratum causing end bearing to carry a major portion of the load.

The Consultant's report has evaluated the load bearing capacities and settlement considerations in some detail and reference should be made to their tabulated calculations.

Piles such as those recommended will not meet refusal. In order to minimize differential settlement and ensure uniform pile driving operations, it would be advantageous to use the same pile length under the entire structure.

The proposed grade line indicates that approach embankment heights will not cause any overstressing of the approach foundations.

contd. p..3.

BORROW AND GRANULAR MATERIALS

Fine to medium sand is plentiful in the immediate vicinity of the structure. The limits of the proposed structure contract are not certain at this time, but it is not expected that much earth grading will be included for the approaches.

Material suitable for concrete aggregate is situated a few miles east of the site.

CONSTRUCTION FEATURES

The new structure is on revised location and the existing structure will not be disturbed during construction.

Protection against erosion on the west bank or at a possible centre pier will probably take the form of piling rather than rip-rap.

Dewatering at piers and abutments will be a problem in the sand which extends down to about 20 feet below the water level in the river. Short sheet piling, left in for protection against scour or ice, may be a solution.

Access to the site is readily available at both approaches.

RECOMMENDATIONS

1. It is recommended that this contract be called as granular borrow with 6" of G.B.C. Class 'A'.

2. The grading will presumably be all fill. Three feet of acceptable granular material should be used in fill section.

A.C. POWELL.
Project Soils Engineer.
Materials & Research Section.

January 5th, 1959.

ACP/TJK/ZW.