

Toronto 2, June 12th, 1958.

Memorandum to
Mr. L. Loch,
Bridge Design Engineer,
Department of Highways.

Re: W.P. - BA 748 - Heyrock Bridge,
Highway # 21, Dist. # 3

W.P. - BA 749 - Blanche River at Judge
Highway # 65, District # 14.

Attached please find above soil reports for your
file.

J. C. McALLISTER
FOR S. McCOMBIE
BRIDGE PLANNING ENGINEER.

Encls.
JCM*IW.

BH 743

TROW, SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

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Project: G108/J202

June 9, 1958.

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

Attention: Mr. J. McAllister

Foundation Investigation
Blanche River Bridge, Judge, Ont.

Dear Sirs:

Enclosed herewith is our report on the foundation conditions and requirements for the structure ultimately proposed for this crossing. This report was prepared by our Mr. K. Peaker, who supervised the field work and the laboratory testing involved in this investigation.

The subsoil underlying this crossing consists of a very deep deposit of the highly plastic and very compressible varved clay which is characteristic of the Great Clay Belt. The general conclusion of the report is that the existing foundations which are terminated in this clay are inadequate to support the final weight of the proposed bridge and appear particularly vulnerable from the point of view of lateral stability. The existing wooden piles have been exposed already by river erosion and early attention appears warranted.

Computations have been made which show how the existing floating foundation can be supplemented by additional piles. However, we favour the more positive solution involving complete support on H piles bearing in the dense sand and gravel found from 100 to 130 feet below river level. We have estimated that each pier will require 28 piles and each abutment will need 12 piles. Regardless of the final foundation decision, all piles must be protected by steel sheet piling, driven about 20 feet below present river bed level and tied at the top.

Riprap protection of the river bank approaches also must be provided.

The opinions and computations of this report are based upon the ultimate loadings suggested to us. If, after your review of the contents, other foundation schemes appear to warrant consideration, we shall be pleased to study the soil mechanics aspects of the suggested proposals.

We thank you for this opportunity to serve you.

Yours very truly,

W. Trow

William A. Trow (P. Eng.)

WAT/lt
Encl.

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

FOUNDATION INVESTIGATION
BLANCHE RIVER BRIDGE
JUDGE, ONTARIO

C108/J202

June 9 1958

Trow Soderman and Associates

TABLE OF CONTENTS

Description of Site	Page 1
Details of Field and Laboratory Work	1
Soil Types Encountered	2
Competence of Existing Foundations	4
Conclusions	6

ENCLOSURES

Appendix	Pages 1 - vi
Summary of Strength Measurements	Table 1
Summary of Other Laboratory Test Measurements	Table 2
Borehole Location Plan	Dwg. 1
Estimated Subsoil Stratigraphy	2
Borehole Profiles	3 - 10
Consolidation Characteristics for Clay Portion of a Varve	11
Vane Shear Strength p.s.f.	12

FOUNDATION INVESTIGATIONBLANCHE RIVER BRIDGE, JUDGE, ONTARIO

This report describes the results of a foundation investigation recently completed for the Blanche River Bridge at Judge, Ontario. Comments regarding the competence of the foundation of the existing structure, together with recommendations for the correction of existing problems are included.

Description of the Site

The site investigated is located at Judge, Ontario, on Highway No. 65, approximately 12 miles east of New Liskeard. The site is presently occupied by approach abutments and piers for the future bridge spans. These abutments and piers are spanned by a temporary Bailey Bridge which has been in place since 1954. The piers for the future bridge are supported by wooden piles, which according to pile driving records were driven to depths ranging from 55 to 90 feet below low water level. The piles supporting the two centre piers are protected by rock-filled wooden cribs resting on the river bottom, while the piles supporting the approach abutments are unprotected.

At present the underside of the east approach abutment has been eroded, exposing the piles. The cribwork on the centre piers is in fair to good condition with the top portion in the poorer state of repair. The crushed rock fill in the cribs has settled or has been washed away so that a gap of approximately 9 inches exists between the fill and the underside of the abutment. The portion of the untreated piles made visible by the exceptionally low water are in good condition with no sign of rot present. The concrete abutments and piers appear excellent with only occasional hair cracks.

The site is surrounded by farm land on all sides with very few trees present. Rock outcrops are visible 3 miles to the east and west of the site and a gravel ridge runs north and south some 300 yards to the east of the river's edge. The river flows in a north-south direction at the bridge crossing and is generally of a fast flowing nature, with the surface flow, at the time of the investigation, estimated at 4 to 6 miles per hour.

Details of Field and Laboratory Work

The field work associated with this investigation was carried out from April 22nd to May 12th. Borings were made with the conventional type diamond drill rig, adapted for soil sampling. Six boreholes and eight dynamic cone penetration tests were located as shown on drawing No. 1. The boreholes were advanced by alternately driving and washing out 2-7/8-inch O.D. casing, while the dynamic cones were driven with an energy equivalent to that used for the standard penetration tests. All

the borings, with the exception of No.1, were made from a floating raft anchored in the river. They were advanced until a dense stratum was encountered approximately 100 feet below the river surface. At this point, the dense material was proved for a depth of 5 to 10 feet or until refusal to washing and driving resulted. An attempt to obtain the elevation of bedrock in borehole No.8 was hampered by sand running into the casing. Because of this running condition, and the extremely dense nature of the sand, the hole was not carried to bed rock, but was stopped 132 feet below the river surface.

Undisturbed samples of cohesive material for shear strength tests were taken with 2 inch I.D. thin-walled Shelby tubing. These samples were sealed to prevent changes in moisture content and transferred to the laboratory for testing. Samples were taken at 5 and 10 foot intervals in boreholes Nos. 1 and 3, but the sampling interval in the remaining boreholes was increased to 20 or 30 feet because of the uniformity of the subsoil. Field vane measurements of the in-situ shearing resistance were obtained for confirmation of laboratory strength test results. Sample numbers and the depths at which they were taken are noted on the borehole logs included at the end of the report.

Elevations were established by the Ontario Department of Highways and are referenced to the geodetic bench mark. The assumed elevations, also in use by the District Office, may be obtained by subtracting 107.37 from any geodetic elevation. The borehole locations were laid out using a steel tape and were referenced to the existing abutments and piers as shown on drawing No.1.

Upon arrival at the laboratory each sample was examined and classified. Moisture content determinations were carried out on the clay portion of most samples and Atterberg Limit tests were conducted to establish the plastic range of the clay. Representative undisturbed samples were tested at in-situ pressures to determine the apparent cohesion of the clay and hence to appraise the capacity of the existing piles. Two representative samples were subjected to consolidation tests, which information is required for the prediction of magnitude and rate of bridge settlement. Test results are recorded on the summary sheets included as Table Nos. 1 and 2 and on Drawings Nos. 3 to 12.

Soil Types Encountered

A detailed description of the soil types encountered in each boring has been presented in Drawings Nos. 3 to 10. The subsoil stratigraphy across the river has been estimated on the basis of this information and is shown in Drawing No.2.

Reference to Drawing No. 2 indicates that the predominant soil type is a very thick deposit of varved clay consisting of alternate layers of medium stiff, very plastic, fissured clay and fine silt. In

nearly all cases the clay phase of each varve is the larger, ranging from 1/8 inch to 2 inches in thickness, while the silt layers range from 1/8 to 1 inch in thickness.

The shear strength of the varved clay was found from laboratory tests to vary between 600 and 900 p.s.f., while field vane tests place the shear strength between 560 and 2000 p.s.f. The discrepancy between the two methods of measurement was most apparent in samples taken below a depth of 40 feet and is attributed to unavoidable disturbances in sampling at this depth, and in the handling of the silt layers immediately prior to test. The field vane results are believed to provide a more accurate indication of actual shear strength and the average value of 1000 p.s.f., used in subsequent calculations, is based upon these field measurements.

The Atterberg limit tests on the clay phases of this deposit indicate a liquid limit of approximately 80% and a plastic limit of the order of 27%, which values conform to the plasticity encountered during investigations at Earlton and in Timmins, Ont. The moisture content is slightly below the liquid limit and decreases somewhat with depth.

Two consolidation tests were performed on the clay portions of this varved soil. The results of these tests show that the clay is highly compressible and very impermeable, with a compression index of 1.36 and a coefficient of consolidation of the order of 0.09 sq.ft. per day. Of particular significance are the measurements of preconsolidation indicated in these tests. In hole 1, the sample from El. 500 ft. has an estimated preconsolidation pressure of 5700 p.s.f. which is about 1000 p.s.f. greater than exists at this location at the present time. This excess pressure is equivalent to 20 feet of clay having a submerged weight of 50 p.c.f., which loading would apply if the original bed of this lake deposit was at elevation 606 feet before the present river cut its channel to present levels. A similar situation was noted in the test on the sample from elevation 494 feet in hole No.3. Here, the present overburden pressure below existing river bed level is approximately 3700 p.s.f. but the sample exhibits a prestress of 2100 p.s.f. in excess of this value. This is equal to the weight of 42 feet of submerged clay which, again, is equivalent to an original surface elevation of 607 feet. The general ground surface of the flat plain, of this broad valley, does not differ significantly from this estimated elevation.

The trend of vane shear strength versus depth shown in drawing No.12, also suggests an original ground elevation of this order. This fact provides additional confirmation of the reliability of these field strength measurements. Further evidence that the present clay has been prestressed is provided by the measurements of moisture content and plasticity. In all samples the moisture content was below, and at depth, well below, the liquid limit. The moisture content of the softer but exactly similar clay from the Earlton area was well above the liquid limit.

The lower moisture content at this Blanche River site could be only the result of consolidation under overburden weights greater than exist at the present time.

Underlying the layer of varved clay is a dense grey fine to coarse sand and fine to coarse gravel. Blows in the dynamic cone test indicate that stones of medium size are present, and samples obtained from the split-spoon revealed sand and gravel in most sizes. Attempts to penetrate this stratum were thwarted by its dense and stoney nature. Large displacement piles of any type would meet refusal in the top few feet of this material while steel H piles may penetrate for some 5 to 10 feet in all areas but borehole No.1. At this location, a dynamic cone was driven with great difficulty to elevation 430.9. Attempts were made to drive a rod at the bottom of hole 8, but it was not possible to pass elevation 442.3. Thus in the vicinity of the west abutment, H piles may penetrate 15 to 20 feet into this stratum.

The medium brown sand encountered at the top of borehole No.1 was the only sand found at the surface. This sand is of a loose nature and is probably the result of recent river deposition. The upper portion may have resulted from erosion of nearby fill material.

Competence of Existing Foundations

According to preliminary approximate estimates, provided by Mr. Locke of the Bridge Design Department, the present light Bailey Bridge loading at this crossing will be replaced by a much heavier structure transmitting loads of the order of 700 tons to each pier and about half this value to the abutments. Before the existing floating pile foundations can be considered to be safe for this loading proposal, they must be acceptable in terms of ultimate stability, long term settlement, and loading on each individual pile. In this latter regard, a value of 15 tons appears to be the maximum acceptable limit for timber piles in D.H.O. design. It conforms closely to the requirements of the Boston Building Code and seems a safe limit considering that the piles are spliced at shallow depth below the present river bed.

Calculations of stability and anticipated settlement are presented in the Appendix of this report. Reference to these computations shows that estimated total pier and abutment loading inclusive of existing concrete and rock-fill is 1354 tons and 593 tons respectively. The estimated safe bearing values for the pile groups under these points of support are 2238 tons and 1628 tons respectively. These values incorporate a factor of safety of three against actual foundation failure and they represent the resistance which each pile group develops both in end bearing and by friction around the perimeter of each group. It assumes a shearing resistance for the clay of 1000 p.s.f. and an average pile length below river bed of 50 feet. A comparison of the estimated bridge loading with this computed safe capacity shows that the factor of safety

against actual failure is well in excess of three. However, the load per pile under the ultimate pier and abutment loads anticipated will be of the order of 32.3 tons and 20.5 tons respectively, values well in excess of the 15 ton limit indicated above. In order to reduce the load to 15 tons and still utilize the present pile system, 49 additional units must be added to each pier and 11 piles to each abutment. A suggested arrangement for these new piles at the pier locations is indicated in the Appendix of this report. It will involve a considerable enlargement of the existing concrete work.

To be effective, these additional piles should be driven to the same depths as those that presently support the bridge, or to approximate elevation 510 feet. Since splices will be required, arrangement should be made for joints much farther below river level than applies at present.

The long term settlements associated with the utilization of the existing pile structure has been estimated to be 10.4 inches for the centre piers, 5 inches for the east abutment and $7\frac{1}{2}$ inches for the west abutment. Because of the very low permeability of the varved clay, this settlement should continue for at least $17\frac{1}{2}$ years at the pier locations and 23 years at the west abutment. One-half of these values would be complete in 4 years and $5\frac{1}{2}$ years respectively. If the foundation is enlarged to accommodate the additional piles and hence keep unit loads at 15 tons, the estimated pier and abutment settlements range from 3 to $5\frac{1}{2}$ inches respectively. This smaller degree of settlement is accounted for not only by the lower unit loading, but also by the fact that most of the bearing stress represents recompression of the soil back to the state which existed prior to the formation of the present river. The settlement should proceed at the same rate as was computed for the existing group.

The foregoing summary indicates what can be done with the existing floating foundation or some modification to it. With this arrangement complete freedom from settlement is not obtainable. In addition, some doubt may still exist concerning the competence of the splice connections of the pile particularly when they will be required to accommodate the thrust of moving ice during spring break-up. It is understood also, that some splices separated during installation of the present piles and that the affected piles were replaced. Since the existing splices are located at shallow depth below present river bed, their ability to withstand lateral thrust will be much less than would apply for joints at greater depth. In view of these qualifications it may be well to consider the use of H piles end-bearing in the underlying sand and gravel.

Small displacement steel H piles should encounter refusal after about 5 to 10 feet penetration into this sand stratum although somewhat greater depths may apply in the vicinity of the west abutment. Assuming 50 tons per unit, approximately 12 piles would be required to support the entire load of each abutment and 28 piles would be needed under the piers.

The piles could be driven at close spacing of the order of $2\frac{1}{2}$ to 3 feet centres and hence could be grouped more closely around the reaction points of each pier. The average length of pile at each pier location will be of the order of 115 feet below elevation 581 feet, the surface of the river. Piles 130 feet in length will be required for the west abutment, and about 100 feet in length under the east abutment. A combination of existing floating piles and fewer end-bearing H piles cannot be considered at this site because, with settlement, all load will be taken in end-bearing.

Regardless of the method used to strengthen the pier and abutment foundations, adequate protection from river scour and erosion must be provided. At present no protection, except for fill material, is in existence at the abutments. The east abutment has been undercut by river erosion and requires immediate attention. The most satisfactory method of protecting the abutments and the piers is with the use of steel sheet piles. Provided that these piles are tied at the top, a penetration of 20 feet below river bed level should be sufficient, since existing scour seems to be limited to the top 5 feet of the river bed. The interior of this steel-sheeted crib work should be topped-up with coarse gravel or crushed rock which should be stabilized with cement grout. The approach fill to each abutment should be protected with graded rip-rap.

Conclusions

The conclusions concerning the installation of a permanent, safe foundation for the piles and abutments at this site, can be summarized as follows:

(1) If bridge loads of 350 tons, live and dead load per reaction, are used, and the load per pile is to be held to 15 tons, both the existing pier and abutment foundations are inadequate. The ultimate load per pile with the existing foundations will be of the order of 32 tons and 20 tons on the piers and abutments respectively. This unit load estimate for the piers is somewhat too severe because it includes all of the weight of the rock cribs. However, a large proportion of this crib weight must be carried by the piles at points just above river bed level.

(2) Considered from the point of view of ultimate stability, the existing pile group is quite competent to carry the anticipated loading. However, the use of a 15 ton per pile limit appears to be wise considering the inherent weakness of the pile joints, particularly when subjected to lateral ice pressure during spring break up. In addition, the reduction in unit loading will tend to minimize both differential and total settlement considerably.

(3) In order to satisfy the 15 ton per pile limit, 49 additional units will be required at each pier and 11 piles must be added under the

abutments. Considerable strengthening of the existing concrete work will be required to accommodate these piles. Long term settlement, with this foundation revision, will be reduced to values ranging from 3 to 5½ inches. In order that these additional piles supplement the existing units, they must be driven to the same depth, which is approximately elevation 510 feet.

(4) A more positive solution to the foundation problem is to support the entire weight of the piers and abutments on steel H piles driven to refusal. This proposal would ensure a good foundation and remove any doubts concerning the condition of the existing wooden piles. Assuming each steel bearing pile has a capacity of 50 tons (12"BP@53#/ft.) 28 piles would be required at each pier location and 12 piles would be required under each abutment.

(5) Settlements associated with either corrective procedure are well within the limits allowable for this type of structure.

(6) Steel sheet piles should be placed around the centre piers and in front of the approach abutments. In this way, all foundations will receive adequate protection from river erosion and scour. The sheet piles may be stopped 20 feet below present river bed level.

(7) Heavy graded rip rap over the approach fills to the embankments will help to protect these areas during periods of high water.

KP/lit
June 9, 1958.
C108/J202



K. R. Peaker
Kenneth R. Peaker (P. Eng.)

APPENDIXA. Ultimate Capacity of Present Foundations

$\gamma' d$ = submerged weight of soil to the pile tips (the submerged weight has been used to account for the weight of the wood piles)

C = shear strength p.s.f. Average value from field vane tests = 1000 p.s.f.

P = outside perimeter of pile group in feet

l = effective depth of pile group in feet = 50 feet

S.F. = safety factor = 3

A = area of pile group in square feet

N_c = bearing capacity factor = 9

(1) Approach Abutments

$$\text{Load taken by skin friction} = \frac{P.C.l}{S.F.} = \frac{86 \times 1000 \times 50}{2000 \times 3} = 718 \text{ tons.}$$

$$\text{Load taken by end bearing} = A(\gamma' d + \frac{9C}{S.F.}) = \frac{330}{2000} (50 \times 50 + \frac{9 \times 1000}{3}) = 910 \text{ T.}$$

Total = 1628 tons.

(2) Centre Piers

$$\text{Load taken by skin friction} = \frac{P.C.l}{S.F.} = \frac{106 \times 50 \times 1000}{2000 \times 3} = 883 \text{ tons.}$$

$$\text{Load taken by end bearing} = A(\gamma' d + \frac{9C}{S.F.}) = \frac{492}{2000} (50 \times 50 + \frac{9 \times 1000}{3}) = 1355 \text{ T.}$$

Total = 2238 tons.

.....

B. Existing loads on Present Foundations(1) Abutments

Estimated weight of concrete = 243 tons

Estimated weight of existing Bailey Br. = 15 tons

Total 258 tons

(2) Piers

Estimated weight of concrete = 189 tons

Submerged wt. of 580 cu.yds. crushed rock

fill in crib = 465 tons *

Estimated wt. of existing Bailey Bridge = 30 tons

Total 684 tons

* Determined from Field Inspector's log; appears high.

(ii)

C. Settlements due to Present Foundations

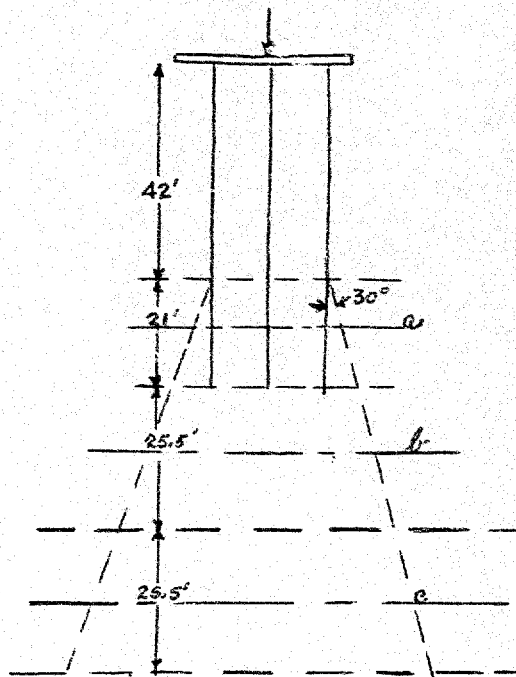
- S = settlement (inches)
 P_o = present overburden pressure (Ksf)
 Δp = change in pressure due to loading (Ksf)
 P_c = max. overburden pressure to which clay has been subjected since deposition
 d = depth of layer consolidating (feet)
 e = void ratio (silt ~ 0.8) (clay ~ 1.8)
 C_c = compression index (silt assumed = .15) (clay = 1.36)
 (clay recompression C_c , = .28)

$$S = \frac{d C_c}{1+e} \log_{10} \frac{\Delta p + P_o}{P_o} \quad \text{below existing state of prestress or preconsolidation } P_c$$

$$S = \frac{d C_c}{1+e} \log_{10} \frac{\Delta p + P_o}{P_c} \quad \text{beyond existing state of prestress}$$

Settlement of West and East Abutments

$$P = 258 \times 2 = 516 \text{ Kips}$$



Calculation for West Abutment

$$\text{Unit load} = \frac{516}{330} = 1.57 \text{ Kips/ft.}^2$$

Assuming 30° load distribution from lower third point (Average embedded pile length = 63 ft.)

$$\text{stress at midpoint a} = \frac{P}{A} = \frac{1.57 \times 330}{1000} = .48 \text{ K/ft.}^2 = \Delta p$$

$$\text{stress at midpoint b} = \frac{P}{A} = \frac{1.57 \times 330}{3520} = .17 \text{ K/ft.}^2 = \Delta p$$

$$\begin{aligned} \text{For a} \\ P_o &= 2625 \text{ K/ft.}^2 \\ P_c &= 3175 \text{ K/ft.}^2 \end{aligned}$$

$$\begin{aligned} \text{For b} \\ P_o &= 3775 \text{ K/ft.}^2 \\ P_c &= 4325 \text{ K/ft.}^2 \end{aligned}$$

where $P_c - P_o$ = weight of soil eroded at this point during formation of present river *

* Assumes original ground level ~ El. 595 ft.

(iii)

Settlement of West and East Abutments -(Cont.)

In both zones a and b, prestress $p_c > p_o + \Delta p$; therefore soil under recompression; use $C_c = 0.28$

Assuming 65% of varved material to be compressible clay and neglecting recompression of silt, which will be negligible:

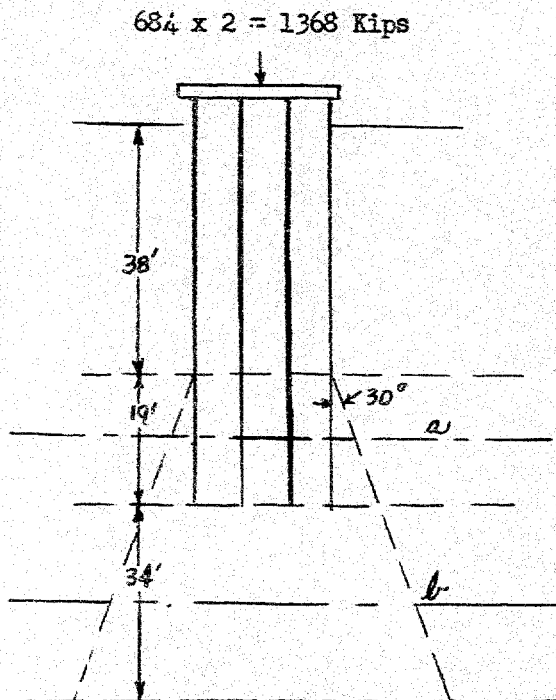
$$\text{zone a} - S = \frac{(21 \times .65) \times 12 \times .28}{2.8} \log \frac{2.625 + .48}{2.625} = \underline{1.2 \text{ inches}}$$

$$\text{zone b} - S = \frac{(25.5 \times .65) \times 12 \times .28}{2.8} \log \frac{3.775 + .17}{3.775} = \underline{.4 \text{ inches}}$$

layer c is negligible

$$\text{Total settlement} = \underline{1.6 \text{ inches}}$$

.....

Settlement of Centre Piers

Assuming 30° load spread

$$\text{Stress at midpoint a} = \frac{P}{A} = \frac{1368}{1070} = 1.28$$

$$\text{Stress at midpoint b} = \frac{P}{A} = \frac{1368}{4160} = .34$$

for a

for b

$$p_c = 2.375$$

$$p_o = 3.700$$

$$p_c = 3.875$$

$$p_c = 5.200$$

recompression of clay layer a

$$S = \frac{(19 \times .65) 12 \times .28}{2.80} \log \frac{2.375 + 1.28}{2.375} = 2.8 \text{ ins.}$$

recompression of clay layer b

$$S = \frac{(.65 \times 34) 12 \times .28}{2.80} \log \frac{3.700 + .34}{3.700} = 0.9 \text{ ins.}$$

$$\text{Total} = \underline{3.7 \text{ ins.}}$$

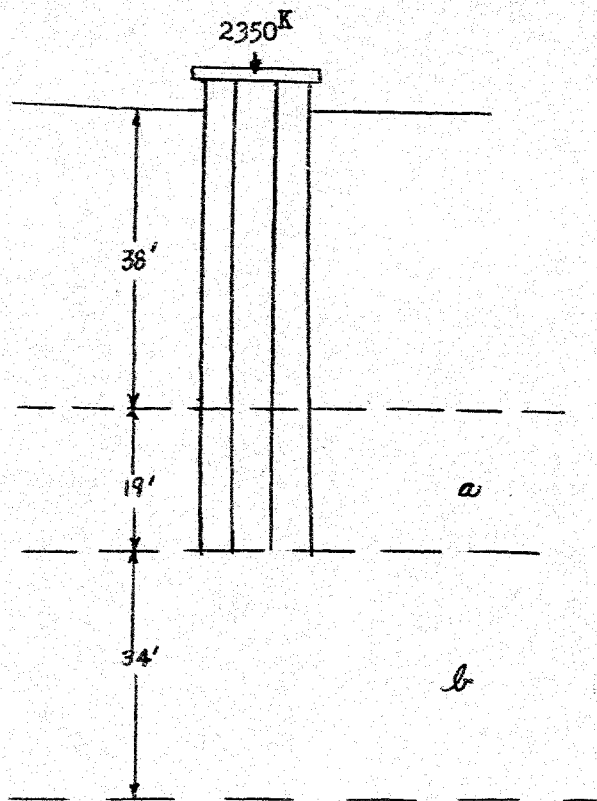
D Analysis for Proposed Bridge

Total loads assuming bridge weight of 350 tons per reaction.

- (1) Load on abutment foundations = 593 tons = 1186 Kips
- (2) Load on centre pier foundations = 1354 T.=2708 Kips

Reference to Section A indicates both piers and abutments are safe.

Settlement of Centre Piers



Using previous assumptions, neglecting live load of 110 p.s.f.

$$\text{Stress at midpoint a} = \frac{P}{A} = \frac{2350}{1070} = 2.20 \text{ K/ft}^2$$

$$\text{Stress at midpoint b} = \frac{P}{A} = \frac{2350}{4160} = .57 \text{ K/ft}^2$$

for a	for b
$P_0 = 2375$	$P_0 = 3700$
$P_c = 3875$	$P_c = 5200$

Settlement of clay in zone a beyond p_c

$$S = \frac{(19 \times .65) \times 12 \times 1.36}{2.80} \log_{10} \frac{2.375 + 2.20}{3875} = 5.2''$$

Recompression of clay in zone a up to p_c

$$S = \frac{(19 \times .65) \times 12 \times .28}{2.80} \log_{10} \frac{3875}{2375} = 3.1''$$

Recompression of clay in zone b

$$S = \frac{(34 \times .65) \times 12 \times .28}{2.8} \log_{10} \frac{4270}{3700} = 1.6''$$

Settlement of silt layers in zone a

$$S = \frac{(19 \times .35) \times 12 \times .15}{1.80} \log_{10} \frac{2.375 + 2.20}{3.875} = .5''$$

$$\text{Total} = \underline{10.4''}$$

Similarly for East and West abutment

Total settlement east abutment 5''

Total settlement west abutment 7½''

E Suggested Modifications for a Maximum Loading
of 15 tons per pile.

Centre Piers

Estimated maximum load = 1354 tons

No. of piles at 15 tons per pile = 91

Present number of piles = 42

Additional required = 49 piles at each pier

These piles can be placed in a ring 2 feet wide around the existing piers as shown below. For a uniform spacing, the piles should be placed at approximately $2\frac{1}{4}$ foot intervals.

Abutments

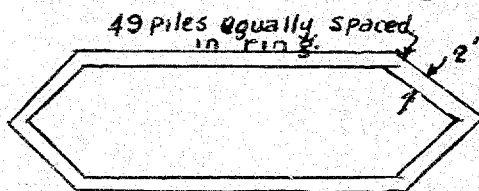
Estimated maximum load = 593 tons

No. of piles at 15 tons = 40

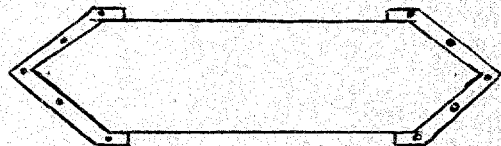
Present number of piles = 29

Additional required = 11 piles each abutment.

These piles could be placed around the ends of the abutments as shown below. In order to obtain symmetry, the total additional piles per abutment is 10.



Suggested arrangement of
additional piles for piers.



Suggested arrangement of
additional piles for abutments.

(vi)

F Summary of Estimated total settlement for each
Type of floating pile foundation

	<u>East Abutment</u>	<u>Centre Piers</u>	<u>West Abutment</u>
Present foundation existing loads	1.6 inches	3.7 inches	1.6 inches
Present foundation under full load	5 "	10.4 "	7½ "
Revised foundation full load at 15 T. per pile	3 "	4 "	5½ "

Estimated duration of Settlement

$$t = \frac{T H^2}{C_v \times 365} \text{ years}$$

where t = the time in years for any degree of settlement under any of the foregoing loading arrangements

T = time factor = .197 for 50% settlement and .848 for 90% settlement

Cv = Coefficient of consolidation ~ .09 sq.ft. per dy.

H = ½ the thickness of compressible soil; most of consolidation will be within and just below the lower third length of the piles.

	<u>East Abutment</u>	<u>Centre Piers</u>	<u>West Abutment</u>
Estimated value of H	20 feet	26 feet	30 feet
50% settlement	2½ years	4 years	5½ years
90% settlement	10½ years	17½ years	23 years

TABLE NO.1

SUMMARY OF STRENGTH MEASUREMENTS IN P.S.F.

Geodetic Elev.Ft.	Hole No. 1		Hole No. 3		Hole No. 4		Hole No. 5		Hole No. 7		Hole No. 8	
	Qu	V	Qu	V	Qu	V	Qu	V	Qu	V	Qu	V
568.0		660 s=3.7										
566.0												
64.0												
62.0		640 s=2.7										
60.0						780 s=6.2						
58.0		800 s=6.7				800 s=4.0		920 s=5.7				
56.0	675					960 s=3.8		1280				
54.0				960 s=4.8						800 s=5.0		
52.0		820 s=3.4		1040 s=4.7						1200 s=5.0		
50.0												
48.0						960 s=5.3		1120 s=4.5			560 s=2.3	
46.0		920 s=4.2				1280 s=3.0		1440 s=3.2			960 s=3.0	
44.0												
42.0				1080 s=6.6								
40.0	650			1080 s=6.6		1280 s=4.5		1440 s=3.8		1040 s=3.5		
38.0						1280 s=3.0		1440 s=3.8		1120 s=4.0	560 s=2.3	
36.0		1200 s=3.7				1520 s=3.4					560 s=2.3	
34.0		1120 s=1.5									1040 s=2.6	
32.0		1120 s=2.0		1280 s=4.0						1200 s=6.0		
30.0				1280 s=3.2						1040 s=3.7		
28.0		1360 s=4.2				1120 s=4.7		1040 s=3.2			960 s=3.4	
26.0		1600 s=3.9				1440 s=6.0		1520 s=3.9			1360 s=4.2	
24.0		1640 s=3.8						1840 s=3.2				
22.0	540											
20.0				1280 s=4.6		1440 s=4.5		1520 s=5.4		1280 s=4.0		
18.0		1240 s=4.0		1360 s=4.3		800 s=2.2		1680 s=3.1		1680 s=4.2		
16.0		1440 s=4.0				880 s=2.2		1920 s=3.4			680 s=2.4	
14.0		1800 s=3.1									800 s=1.3	
12.0				1600 s=5.0	625					1440 s=4.5	1440 s=3.6	
10.0				1760 s=3.7						1760 s=4.5		
8.0		1680 s=4.2				1440 s=3.6		1760 s=4.9			760 s=2.8	
6.0		1600 s=4.0	950			1360 s=3.0		2080 s=3.2			1840 s=5.0	
4.0		2000 s=2.0										
2.0				2000 s=4.1						1760 s=4.0		

TABLE NO.1 - Cont.

Geodetic Elev.Ft.	Hole No.1		Hole No. 3		Hole No. 4		Hole No. 5		Hole No. 7		Hole No. 8	
	Qu	V	Qu	V	Qu	V	Qu	V	Qu	V	Qu	V
500.0	925			2000 s=2.8			1440 s=3.8		1600 s=2.6		840 s=2.6	
498.0		1480 s=5.7					1440 s=3.0				800 s=2.0	
96.0		2240 s=2.2	575				1680 s=2.8	700			1000 s=3.9	
94.0		1840 s=2.3										
92.0				2000 s=4.1			650		1680 s=3.6			
90.0	775			2000 s=2.5					1440 s=2.0			
88.0											960 s=2.1	
86.0		1280 s=2.3									1400 s=2.0	
84.0		1440 s=2.3							1480 s=3.4			
82.0		1440 s=2.3							1840 s=3.1			
80.0	750								1840 s=2.6			
78.0												
76.0												
74.0												
72.0												
70.0												
68.0												
466.0											1520	

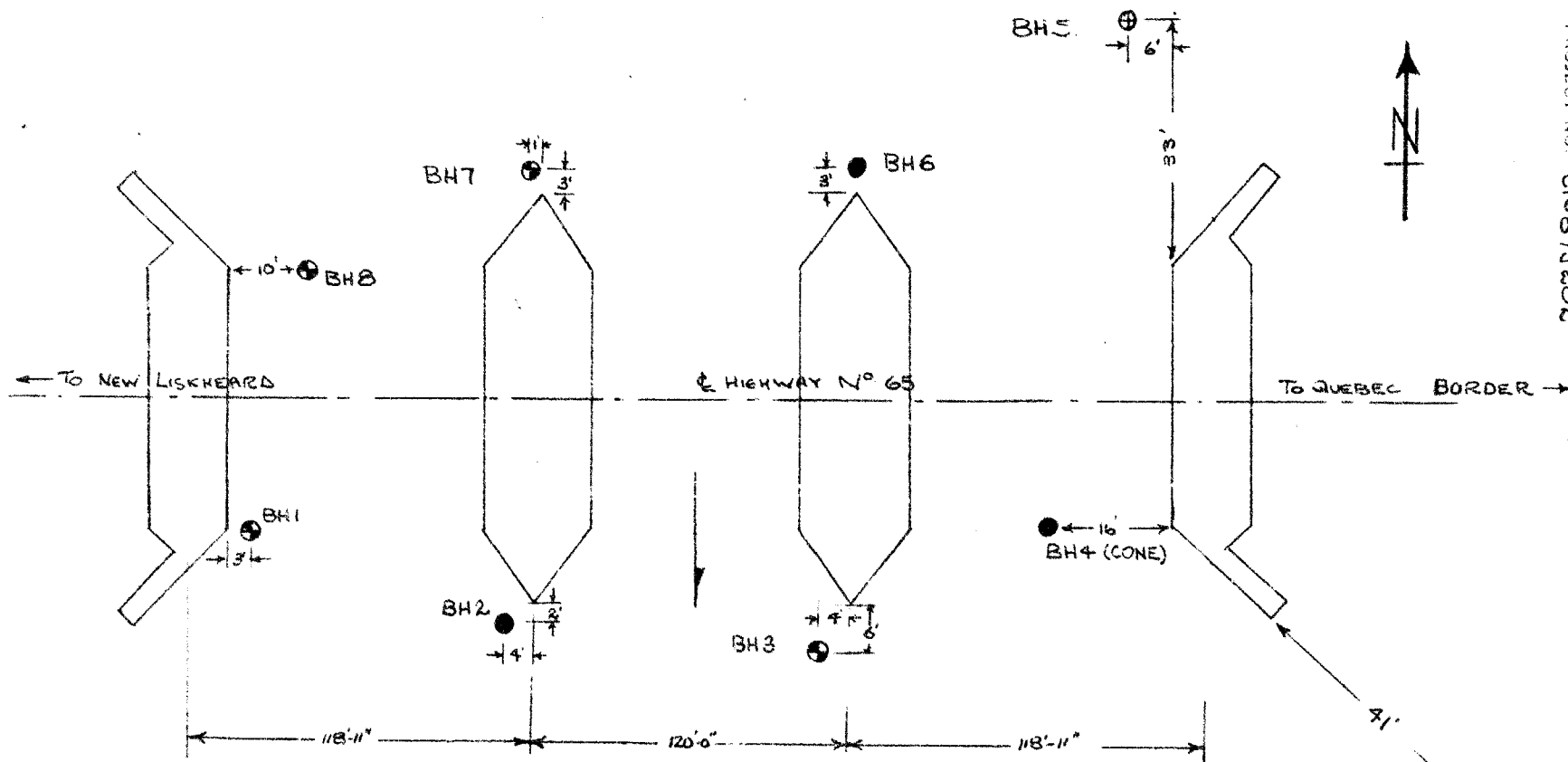
LEGEND: Qu - Undrained triaxial test
V - Vane
s - sensitivity

TABLE NO. 2

SUMMARY OF OTHER LABORATORY TEST MEASUREMENTS

Elev- ation	Natural Moisture Content					Atterberg Limits		Natural Unit Weight				
	% Dry Weight					% Dry Wt.		p.c.f.				
	Hole					L.L.	P.L.	Hole				
	1	3	4	5	7	clay		1	3	4	5	7
564.9	72.5					84.1	23.9 (H1)	108.5				
559.9								113.8				
554.9	75.3	73.9						105.0				
550.9				68.0							108.3	
549.9	75.8					80.8	27.2 (H1)	109.1				
544.9					75.0							
539.9	62.8							108.5				
534.9									103.5			
530.9				67.5				110.0			110.6	
524.8		74.3			50.3		25.1 (H7)	109.3				109.6
519.9	69.8							118.8				
514.8		64.2							108.1			
511.8			71.8	64.3		72.0	28.8 (H5)			103.5	110.6	
504.8		62.6							107.3			
499.9	65.7							109.2				
494.8		53.4				84.6	26.6 (H7)		118.1			107.5
490.8				55.8							118.8	
488.9								117.1				
479.9								115.2				

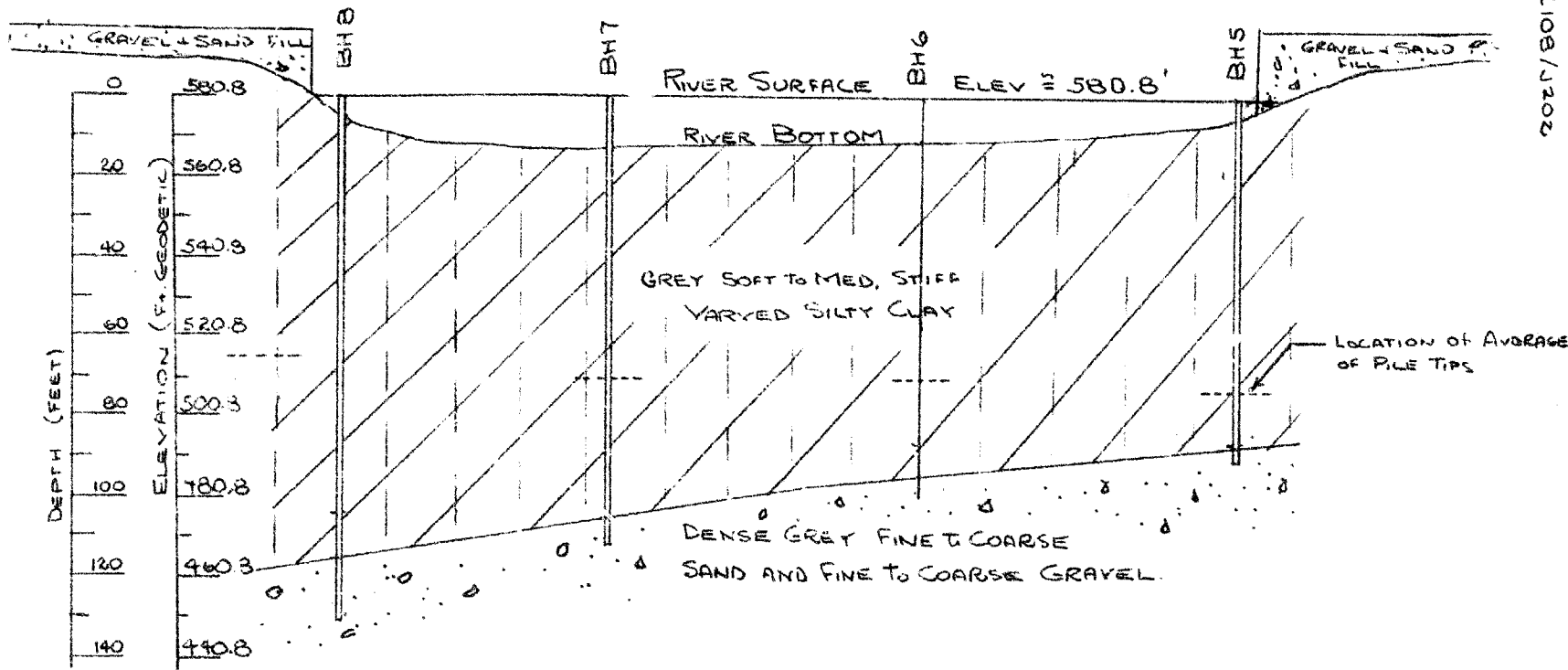
LEGEND: L.L. - Liquid Limit
P.L. - Plastic Limit



BORE HOLE LOCATIONS
BLANCHE RIVER BRIDGE

NOT TO SCALE:

MAY 1958



ESTIMATED SUB-SOIL STRATIGRAPHY

SCALE: HORIZ 1" = 60'
VERT. 1" = 40'

BLANCHE RIVER BRIDGE
HIGHWAY No. 65

MAY 1958

TROW SODERMAN AND ASSOCIATES

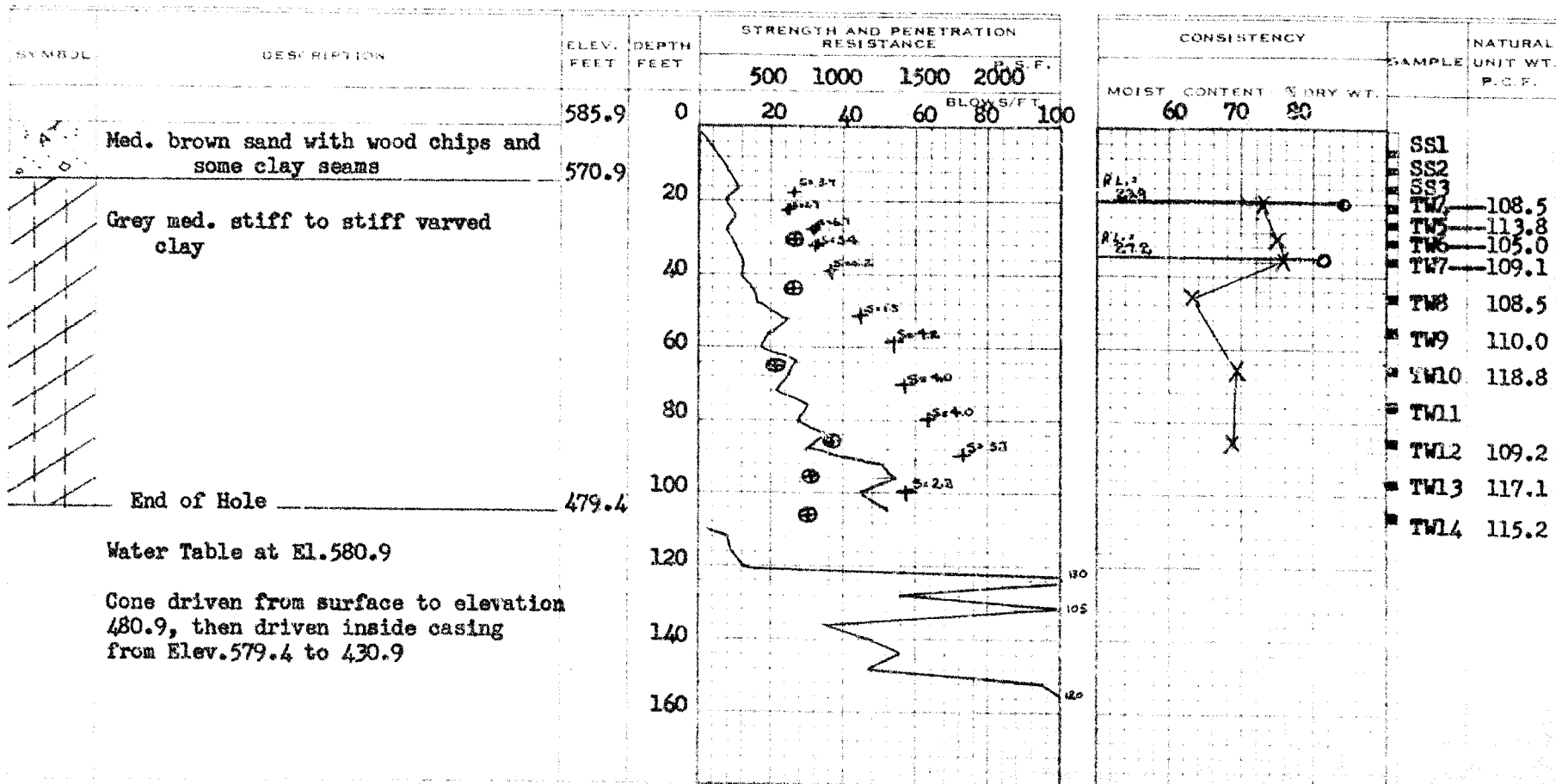
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

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 **Blanche River Bridge**
 LOCATION **Highway 65, Judge, Ont.**
 HOLE LOCATION **See Plan**
 HOLE ELEVATION AND DATUM **Elev. 585.9 Geodetic**

BOREHOLE NO. **1**
 FIELD SUPERVISOR **KP**
 DRILLER **KP**
 PREP. **KP**



PROJECT NO.

C108J202

DRAWING NO.

4

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

Blanche River Bridge

LOCATION Hwy. 65, Judge, Ont.

HOLE LOCATION See plan

HOLE ELEVATION AND DATUM 580.8 Geodetic

BOREHOLE NO. 2

FIELD SUPERVISOR KP

DRILLER MG

SELF. AP

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.	
	Water surface	580.8	0	BLOWS PER FOOT	
	River bottom	565.8	20		
			40		
			60		
			80		
			100		
		468.3	120		

The graph plots penetration resistance in blows per foot against depth in feet. The curve shows a gradual increase in resistance from the water surface down to about 100 feet, after which it levels off slightly. The final data point at 120 feet depth corresponds to a resistance of 108 blows per foot.

[illegible]

TROW SODERMAN AND ASSOCIATES

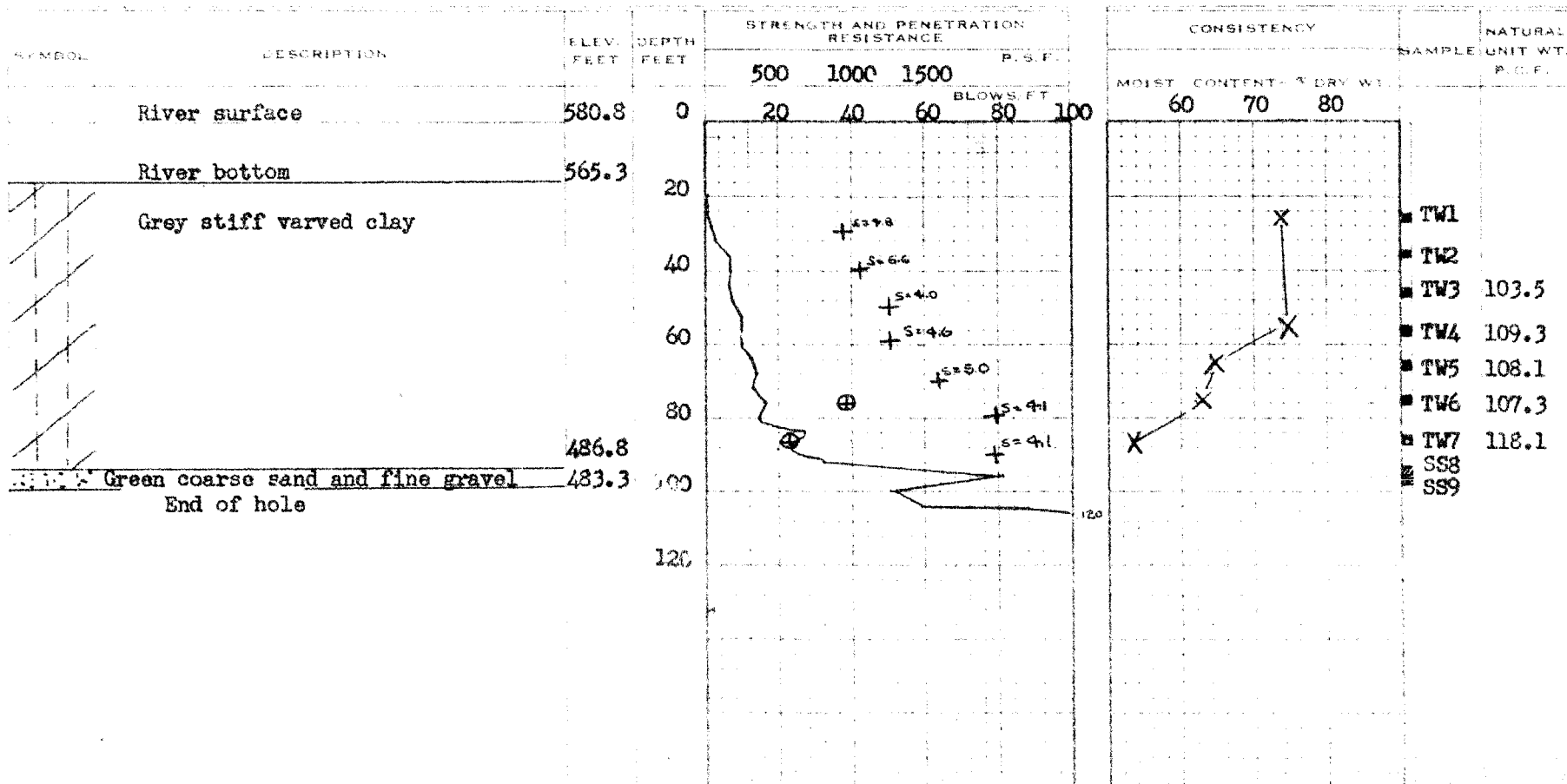
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT **Blanche River Bridge**
 LOCATION **Hwy. 65, Judge, Ont.**
 HOLE LOCATION **See plan**
 HOLE ELEVATION AND DATUM **580.8 Geodetic**

BOREHOLE NO. **3**
 FIELD SUPERVISOR **KP**
 DRILLER **MG**
 PREP. **KP**

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 [SI]
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT



TROW SODERMAN AND ASSOCIATES

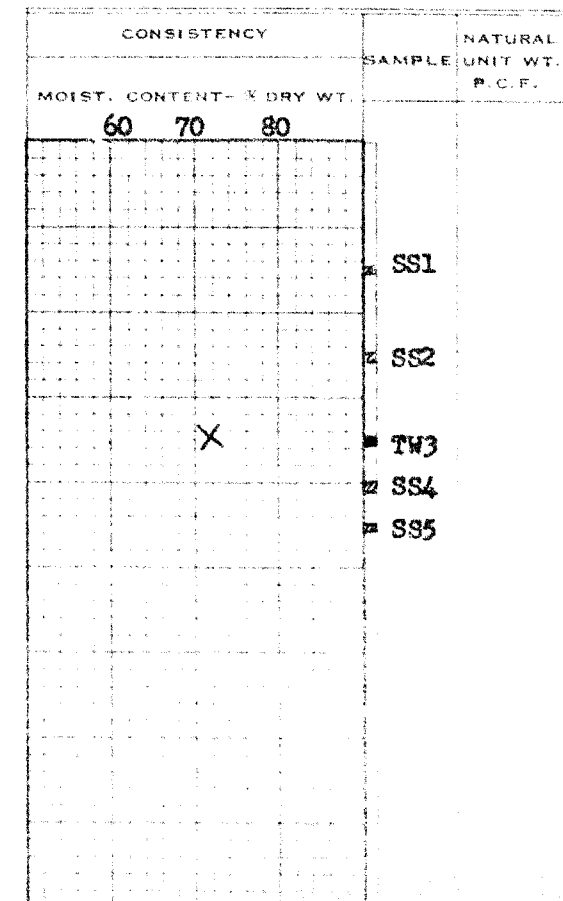
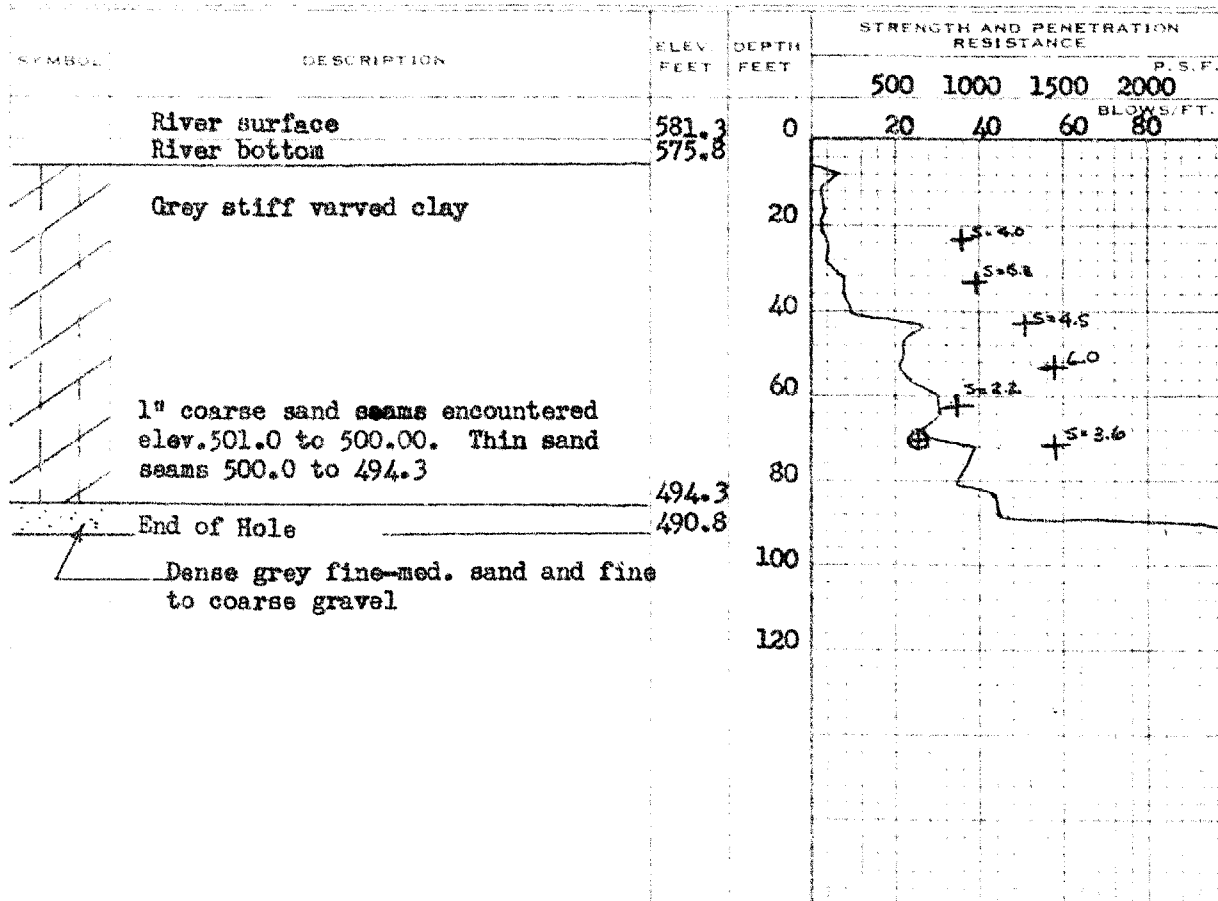
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT **Blanche River Bridge**
 LOCATION **Hwy. #65 Judge, Ont.**
 HOLE LOCATION **See plan**
 HOLE ELEVATION AND DATUM **Elev. 581.3 Geodetic**

BOREHOLE NO. **4**
 FIELD SUPERVISOR **KP**
 DRILLER **MG**
 PREP. **KP**

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



TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

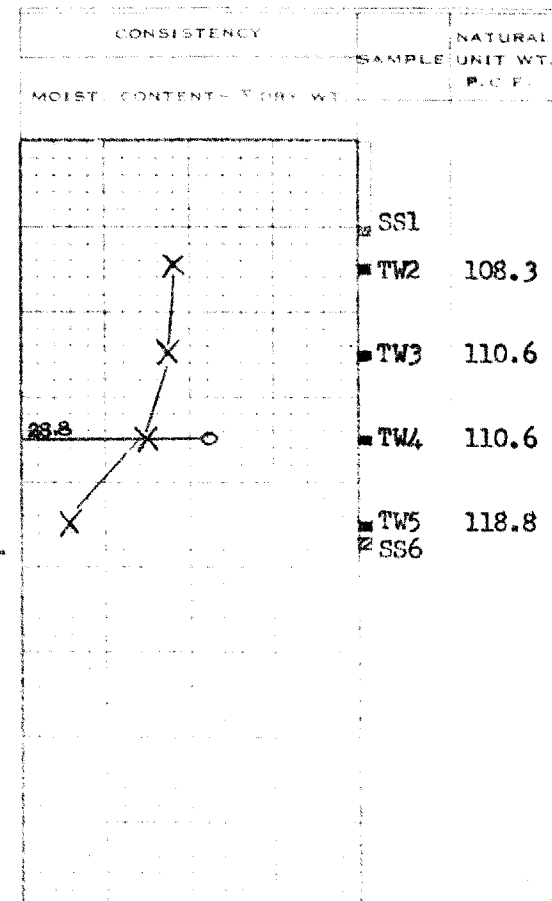
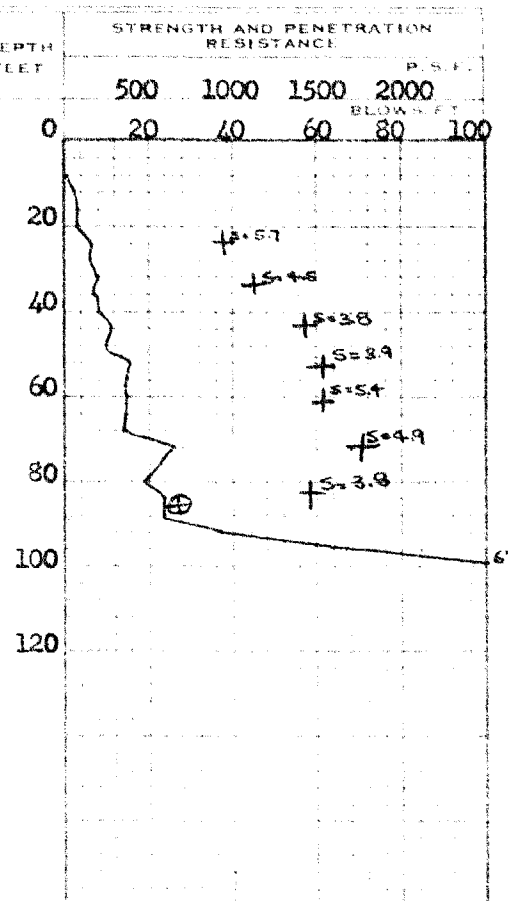
PROJECT Blanche River Bridge
 LOCATION Hwy. 65 Judge Ont.
 HOLE LOCATION See plan
 HOLE ELEVATION AND DATUM 580.8 Geodetic

BOREHOLE NO. 5
 FIELD SUPERVISOR KP
 DRILLER MG
 PREP. KP

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
	River surface	580.8	0
	River bottom	575.3	
	Grey stiff varved clay		
	Thin seams fine sand elev. 485.0 to 489.8		
	Grey fine-med. sand with fine-coarse gravel	489.8	
	End of bore	485.8	



TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT **Blanche River Bridge**
LOCATION **Hwy. #65, Judge, Ont.**
HOLE LOCATION **See plan**
HOLE ELEVATION AND DATUM **580.3 Geodetic**

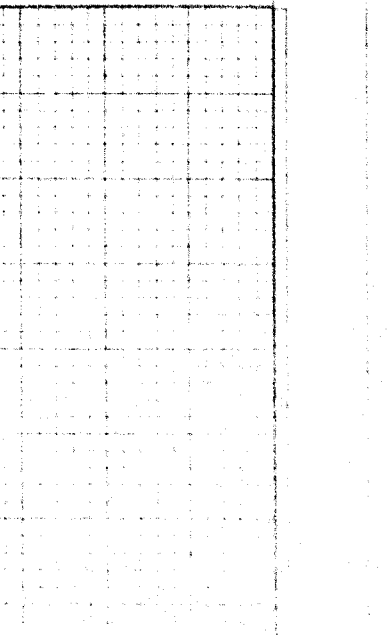
BOREHOLE NO. 6 (Dynamic Cone
FIELD SUPERVISOR KP Only)
DRILLER MG
PREP. KP

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.	
	River surface	580.8	0	20	40
	River bottom	570.3		60	80
			20		
			40		
			60		
			80		
		478.1	100		

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

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT

Blanche River Bridge

LOCATION

Hwy #65 Judge On.

HOLE LOCATION

See plan

HOLE ELEVATION AND DATUM

580.8

BOREHOLE NO.

17

FIELD SUPERVISOR

KF

DRILLER

MG

PREP.

KE

LEGEND

- 2" DIA. SPLIT TUBE

- 2" SHELBY TUBE

- 2" SPLIT TUBE

- 21 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			
				500	1000	1500	2000
				P.S.F.			
				BLOWS/FT.			
	River surface	580.8	0	20	40	60	80
	River bottom	567.8					
	Crushed stone fill	562.8	20				
	Grey stiff varved clay		40				
			60				
			80				
			100				
			120				
	End of Hole	468.3					

Dense grey fine to coarse sand and fine to med. gravel.

NOTE: Unable to drive cone at bottom of casing. Sand running in at bottom of hole.

CONSISTENCY		SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.			
60	70	80	
		X	TW1
X			TW2
			TW3
26.6		O	TW4
			TW5
			SS6

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Blanche River Bridge
LOCATION Hwy. #65 Judge Ont.

HOLE LOCATION See plan

HOLE ELEVATION AND DATUM **Elev. 580.8 Geodetic**

BOREHOLE NO. 8

FIELD SUPERVISOR KP

DRILLER MG

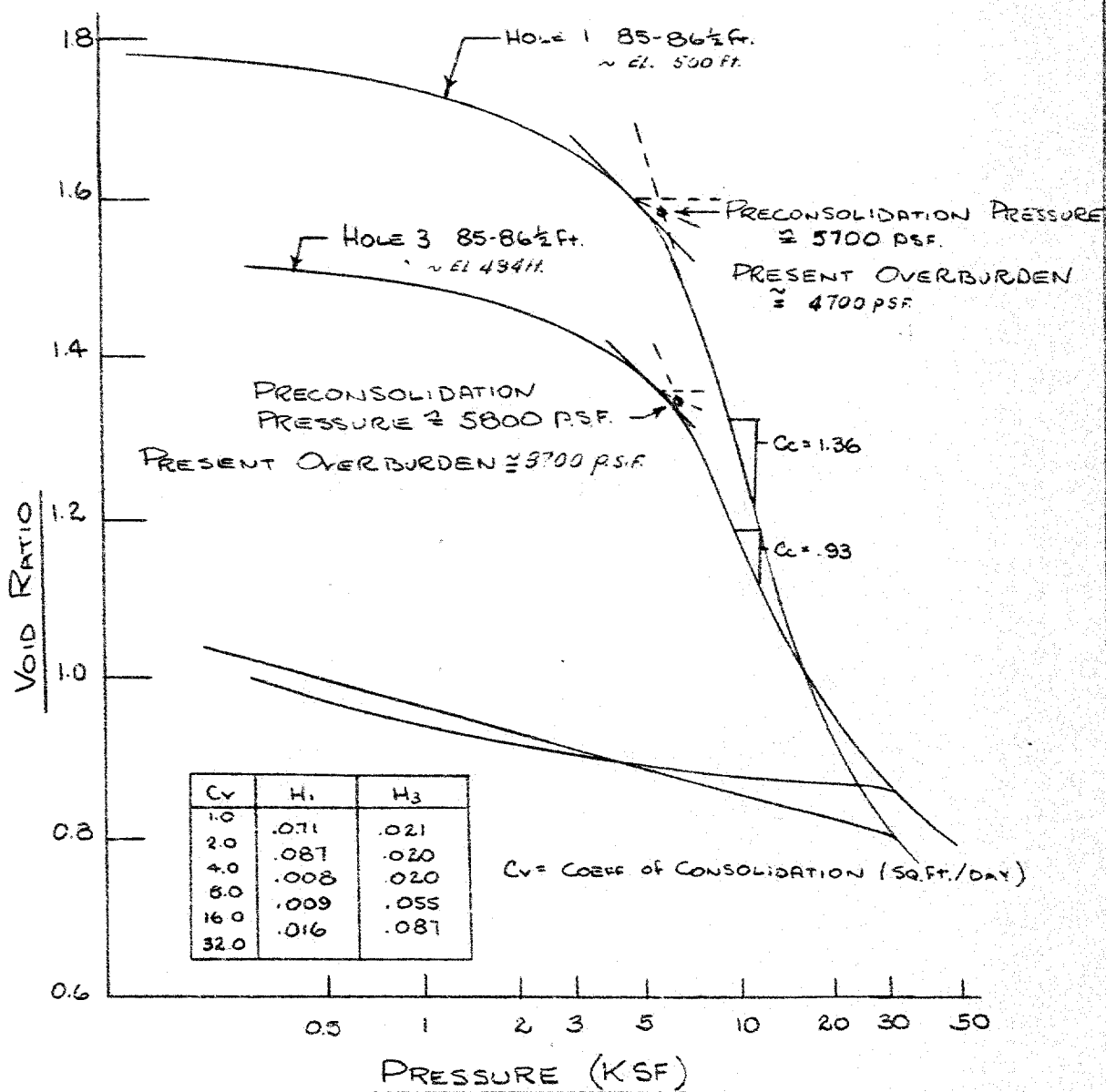
PREF. KP

LEGEND

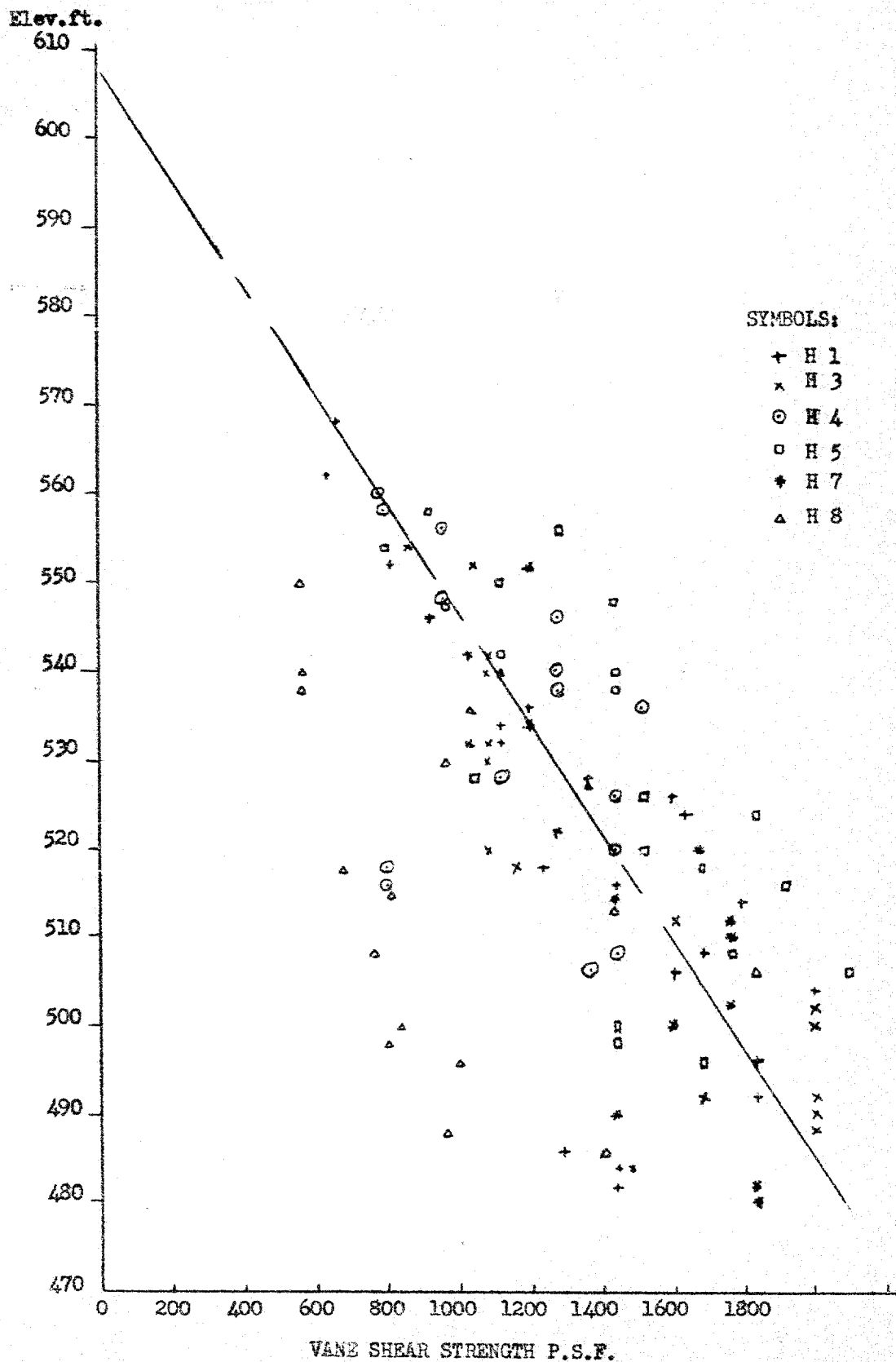
- 2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
CASING
2" SHELBY
1/2 UNCONFINED COMPRESSION [Qu]
VANE TEST [C] AND SENSITIVITY [S]
NATURAL MOISTURE AND
LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
					P.S.F.
	River surface	580.8	0	20	40
	— River bottom	576.3		60	80
	Grey med. stiff to stiff varved clay		20		
	Very thin sand seams noted at elev. 530.8 to 466.8		40		
			60		
			80		
			100		
		466.8	120		
	Grey & black fine to coarse sand and fine to coarse gravel	448.5	140		
	End of Hole				
	NOTE: Artesian head of 3 ft. noted at elev. 464.8				
	Sand running in at bottom of hole.				
	Drove drill rods from elev. 453.3 to 442.3				

[illegible]

BLANCHE RIVER BRIDGE

CONSOLIDATION CHARACTERISTICS
FOR CLAY PORTION OF A VALVE



TROW, SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

W. A. TROW, M.A.S.C., M.E.C.E., P.E.N.G.
L. G. SODERMAN, B.Sc., D.V.C., P.E.N.G.

884 WILSON AVE
DOWNSVIEW, ONT
ST. 8-5921

Project: C108/J202

June 9, 1958.

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

Attention: Mr. J. McAllister

Foundation Investigation
Blanche River Bridge, Judge, Ont.

Dear Sirs:

Enclosed herewith is our report on the foundation conditions and requirements for the structure ultimately proposed for this crossing. This report was prepared by our Mr. K. Peaker, who supervised the field work and the laboratory testing involved in this investigation.

The subsoil underlying this crossing consists of a very deep deposit of the highly plastic and very compressible varved clay which is characteristic of the Great Clay Belt. The general conclusion of the report is that the existing foundations which are terminated in this clay are inadequate to support the final weight of the proposed bridge and appear particularly vulnerable from the point of view of lateral stability. The existing wooden piles have been exposed already by river erosion and early attention appears warranted.

Computations have been made which show how the existing floating foundation can be supplemented by additional piles. However, we favour the more positive solution involving complete support on H piles bearing in the dense sand and gravel found from 100 to 200 feet below river level. We have estimated that each pier will require 28 piles and each abutment will need 12 piles. Regardless of the final foundation decision, all piles must be protected by steel sheet piling, driven about 20 feet below present river bed level and tied at the top.

Riprap protection of the river bank approaches also must be provided.

The opinions and computations of this report are based upon the ultimate loadings suggested to us. If, after your review of the contents, other foundation schemes appear to warrant consideration, we shall be pleased to study the soil mechanics aspects of the suggested proposals.

We thank you for this opportunity to serve you.

Yours very truly,

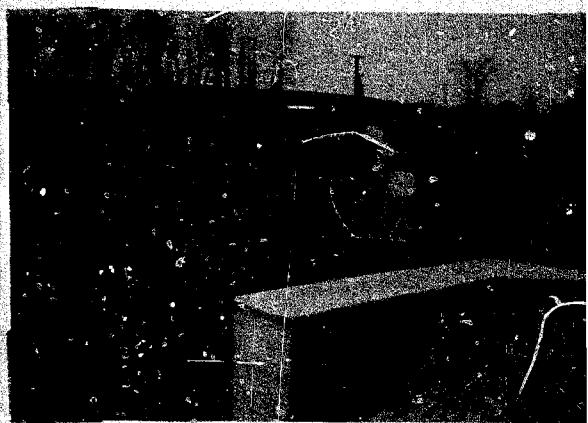
W. Trow

William A. Trow (P. Eng.)

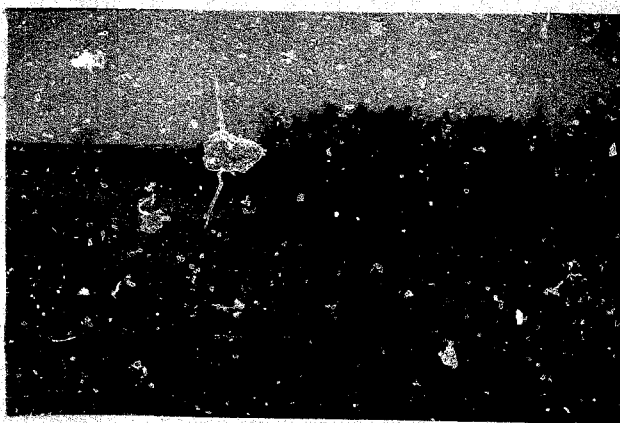
WAT/lt
Encl.



East approach of bridge taken
from west abutment.



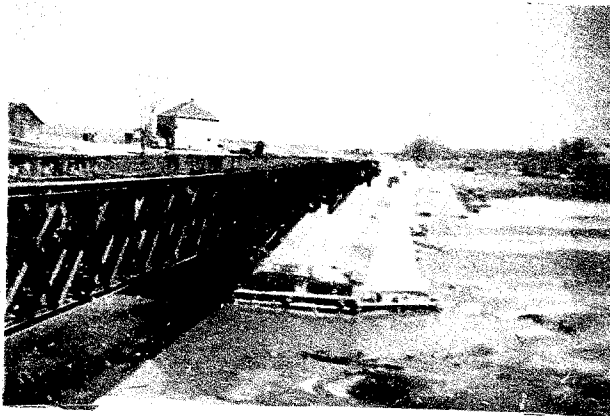
West approach of bridge
taken from east abutment.



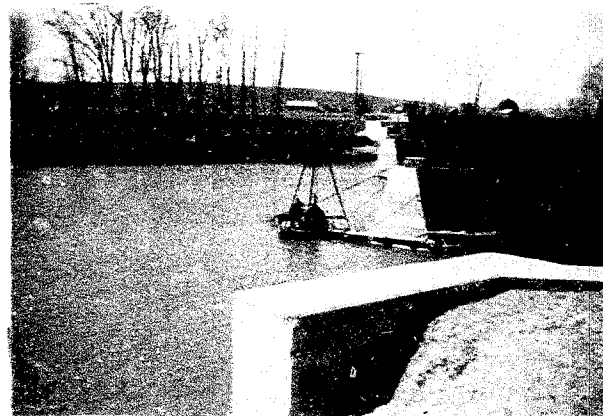
View downstream of old bridge
piers taken from east abutment.



Upstream view from east
abutment.



East approach of bridge taken
from west abutment.



West approach of bridge
taken from east abutment.



View downstream of old bridge
piers taken from east abutment.



Upstream view from east
abutment.

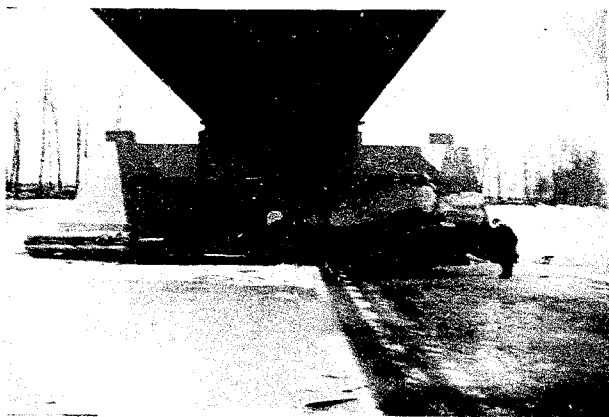


Close up of
centre pier
looking west.

Exposed piles under
east abutment.



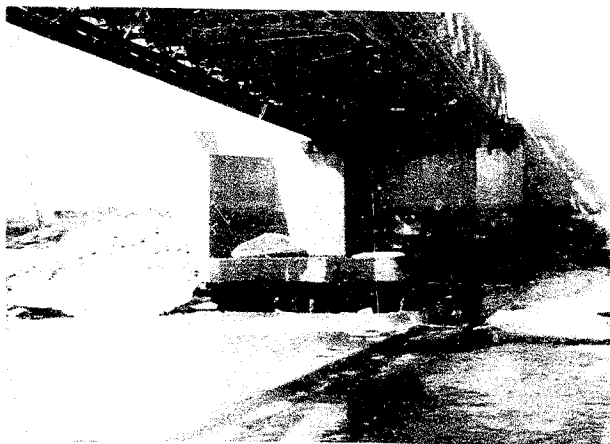
East abutment showing
undercutting of pile
cap.



Close up of
centre pier
looking west.



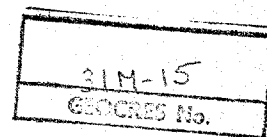
Exposed piles under
east abutment.



East abutment showing
undercutting of pile
cap.



ONTARIO
DEPARTMENT OF HIGHWAYS



Memo to Mr. A. M. Foye, *Date* December 18, 1959.
Bridge Engineer. *Subject* D.H.O. PILE LOADING TESTS --
From Materials & Research Section.

Attention: Mr. S. McCombie.

Re: Blanche River Structure - Hwy. 65,
Contract No. 59-269 - Dist. No. 14.

In response to a request from Mr. B. Davis, Bridge Design Engineer, this Section has carried out static and dynamic pile loading tests to determine the required pile lengths at the above structure. This request was initiated by the District personnel immediately they realized that adequate pile capacities were not being obtained by driving piles to the elevations shown on the bridge drawings.

A visit to the site was made on November 13/59 by - Messrs. K. Peaker, Foundations Office, B. Wilkie and P. McWatt, Bridge Office. Data obtained, confirmed that 'H' piles driven at the West abutment were not obtaining the required final set values until the tip elevation was 412.0' (i.e., 38 feet below required tip elevation shown on contract drawings). In an attempt to obtain acceptable pile capacities at or above elevation 412.0', it was decided to drive and test a modified 'H' pile section. The modification consisted of welding a 12-inch square plate to the base of the pile and lagging the web for a length of 20 feet above the tip. This increases the area of the pile base and serves to make the pile function as a large displacement pile, thereby effecting an increase in load carrying capacity.

cont'd. /2 ...

Four static load tests were carried out on piles driven at the West abutment. Pile No. 26, a modified section, was driven to elevation 459.0', statically tested and then driven to elevation 449.0' and tested again. Pile No. 32, which was a standard 12 BPC 53, was driven to elevation 458.0' and statically tested. Pile No. 29, also a standard 12 BPC 53, was driven to elevation 412.0' and tested. A plan showing the location of the piles tested and the static load test results, is attached to this report.

An analysis of the test results obtained, gives rise to the following conclusions:-

(1) The modified pile section did not show a sufficient increase in load carrying capacity to allow this section to be adopted and founded at or above elevation 449.0'.

(2) The load carrying capacity of standard 12 BPC 53 section, driven to elevation 460.0', is not adequate to meet the design load.

(3) In order to obtain the required design loading for piles at this site, it is our recommendation that the standard 12 BPC 53 sections be driven until a final set of 80 blows/ft. is obtained using a Delmag D 12 Hammer. For construction control purposes, the following criterion is suggested:-

One foot of pile penetration at 80 blows can be considered adequate if this penetration resistance is immediately preceded by three feet of penetration in excess of 50 blows/ft.

The pile lengths recommended in the Consultants' report have been reviewed by this Section, and also by Dr. H. Q. Golder who is an international expert in the foundations field. The

cont'd. /3 ...

obvious discrepancy between the theoretical and actual capacities determined by static load tests is believed due to the unpredictable influence of excess hydrostatic pressures in the underlying stratum of sand. This was also the case at the Big Pic River - fortunately, it was not necessary to resort to electro-osmosis at the Blanche River.

The main points covered in this report have been given to the District personnel, and pile driving is nearing completion. The attached drawing shows our estimated refusal elevation at the East abutment and intermediate pier location.

In conclusion, I should like to point out that there are still a number of unresolved problems in the field of Soil Mechanics and Foundations, and the problem incurred at this site is a classic one. If anyone has any data on pile tests that have been carried out in conditions similar to this, it would be appreciated if such data could be forwarded to the Foundation Section for analyses.



LGS/MdeF
Attach.

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGINEER

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
D. G. Ramsay
G. K. Hunter
R. S. Chapman
E. R. Saint

Foundations Office
Gen. Files.

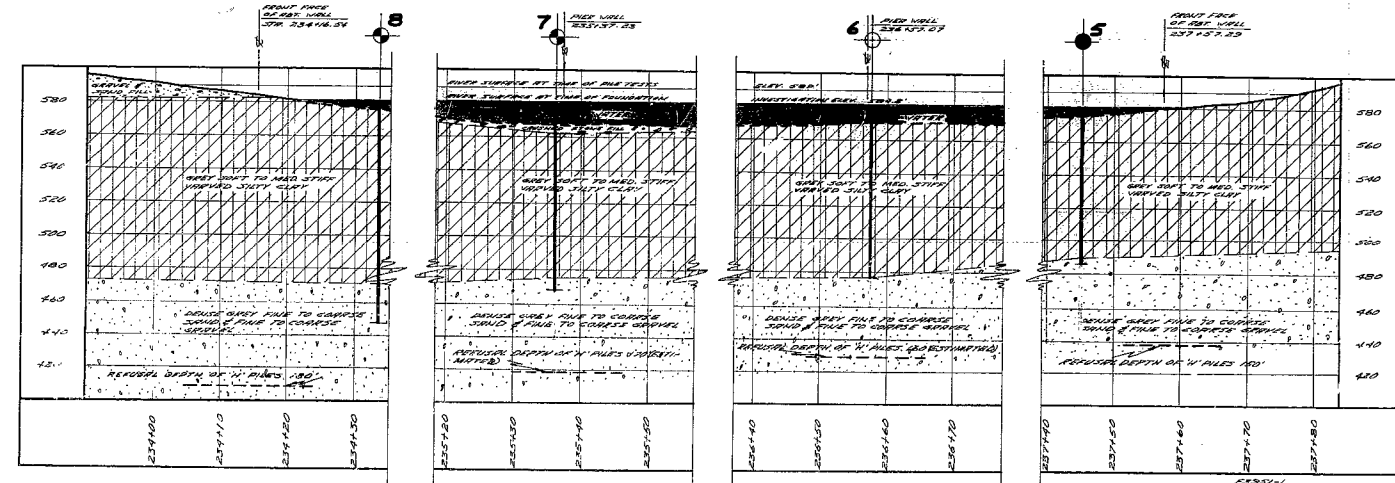
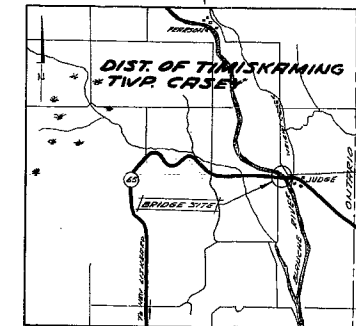
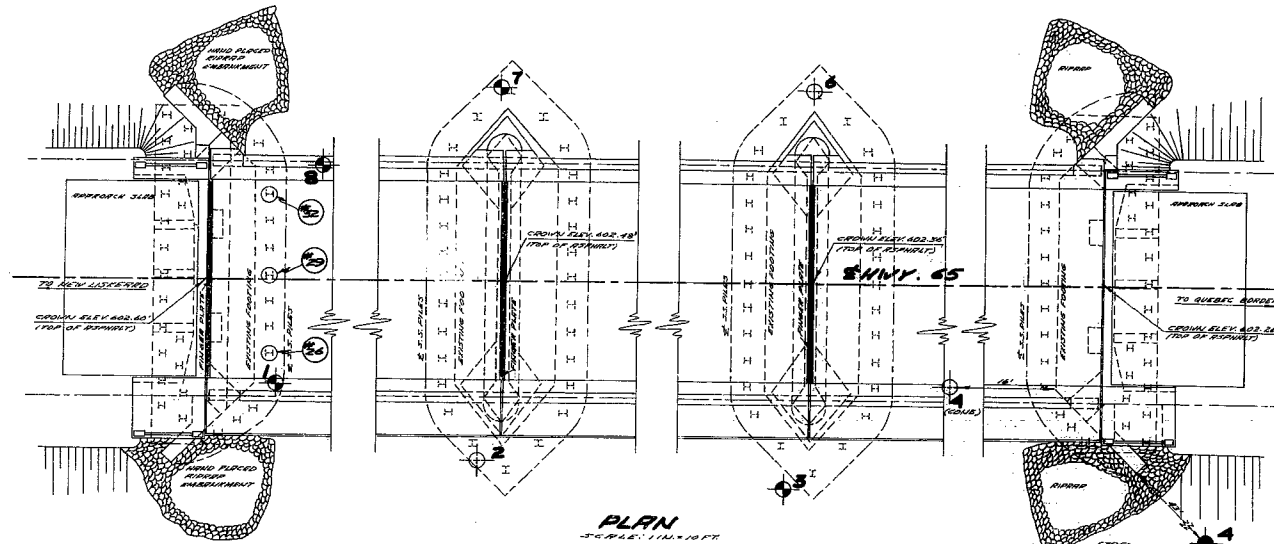
CONT. 59-259

HWY. 65

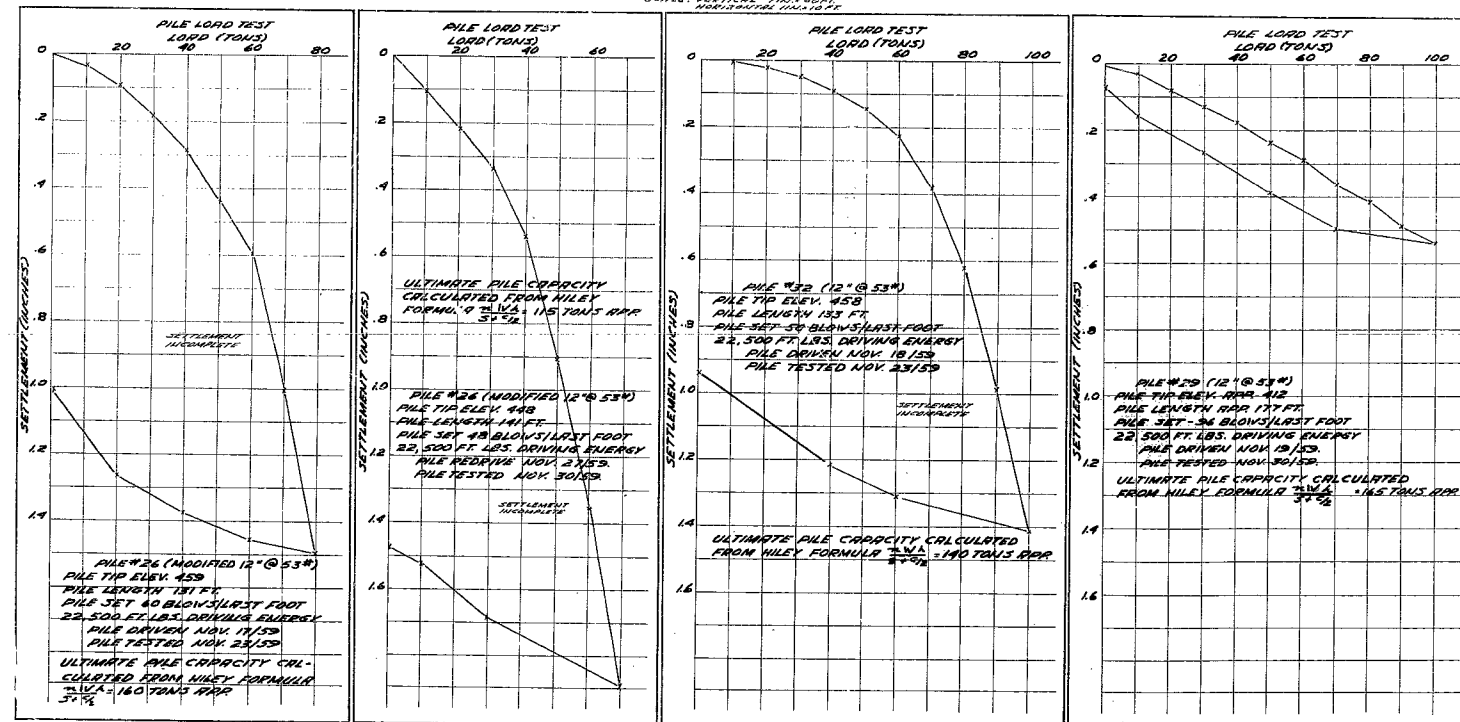
BLANCHE

RIVER

31M-15



LEGEND			
BORE HOLE			
PENETRATION HOLE			
BORE & PENETRATION HOLE			
HOLE NO.	ELEVATION	STATION	DISTANCE FROM PILE
1	585.2	234+12.7	16' 8"
2	580.8	235+34	27' 10"
3	580.8	236+54	31' 10"
4	580.8	237+15	15' 10"
5	580.8	237+45	52' 10"
6	580.8	238+57	29' 10"
7	580.8	235+26	29' 10"
8	580.8	234+34	17' 10"



31M-15

NOTE
THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BORE HOLE LOCATIONS. BETWEEN BORE HOLES THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE AND MAY BE SUBJECT TO CONSIDERABLE ERROR.

CONT. 59-259

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION

BLANCHE RIVER CROSSING

SHOWING POSITIONS & ELEVATIONS OF HOLES

HWY. 100 DISTRICT 10 COUNTY 22

TOWNSHIP 10E RANGE 10E

LOCATION 10E 10E

DRAWN BY: T. HALLORAN CHECKED BY: J. H. HALLORAN

DATE 11 DEC. 59 APPROVED BY: J. H. HALLORAN

SCALE 1/4" = 10' HORIZ. 1/4" = 10' VERT.

W.P. 10-39
F59-1210