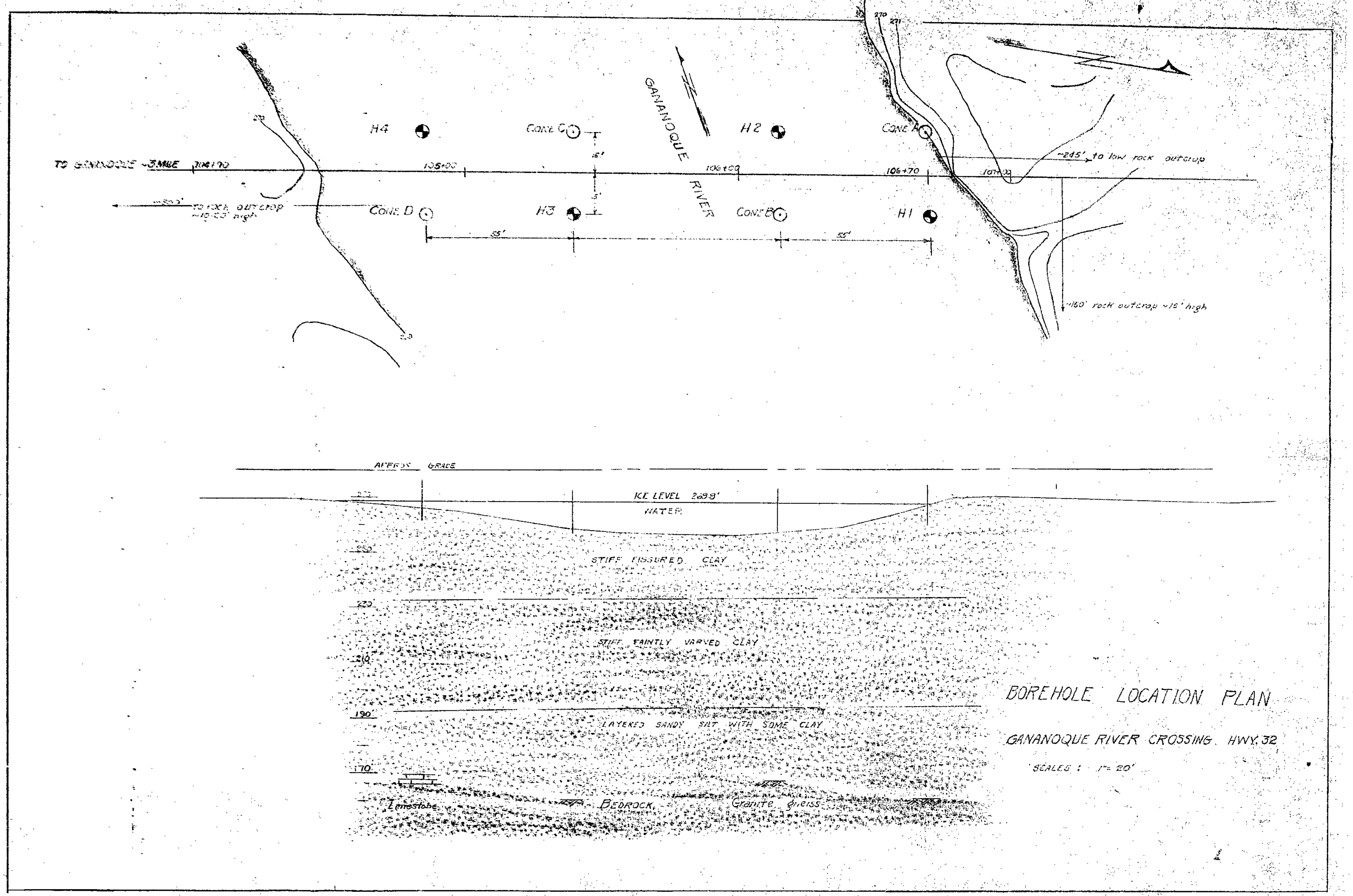
#59-F-239-C
HWY. #32
GANANOQUE
RIVER CROSSING



BOREHOLE LOCATION PLAN

GANANOQUE RIVER CROSSING HWY. 32

SCALES : 1" = 20'

BB3 76-A

WILLIAM A. TROW AND ASSOCIATES

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

W. A. TROW, M.A.S.C., M.E.I.C., P.ENG.

884 WILSON AVE.
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Project: J 343

April 10, 1959.

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

Attention: Mr. K. G. Bassi

Estimated Settlements - Gananoque River Bridge
Highway #32, near Gananoque, Ont.

Dear Sirs:

In conformance with your recent request we have re-examined our estimates of settlement of the above-noted structure using the abutment and pier loads submitted in your drawing D-4309-P1.

The calculations associated with this analysis are attached to this letter for reference purposes. The results may be summarized briefly as follows:

(1) The estimated long term settlement of the bridge piers under a design load of 425 Tons is 3.6 inches.

(2) The estimated long term settlement of the bridge abutments as shown, with embankments approximately 13 feet high and supporting design load of 600 tons, is of the order of 12 inches. This value applies for a group of 60 wood piles driven either 20 feet or 30 feet below river bed level.

(3) The estimated long term settlement of the approach fill, caused by its own weight, is 6.8 inches.

(4) The estimated long term settlement of the abutments, assuming no embankment fill within 20 feet and a simple approach slab spanning the distance between the fill and the abutments, is 6 inches. With this arrangement, the differential movement between the pier and the adjacent abutment will be 2.4 inches, or 1/270 of the span between these two bridge units.

The foregoing estimates of settlement have been based upon the results of three consolidation tests presented in our report of March 16th. Some allowance has been made for the over-consolidated state of the clay, and for the finite dimensions of the pile group. The calculations assume that the design loads for the piers and abutments are effective for a major part of the time. This approach is conservative because some live and impact loading is probably included in these values.

The conclusion to be drawn from these calculations is that the long term differential settlement associated with the design proposal shown in Dwg. D-4309-P1 is much greater than can be tolerated. This undesirable situation results, in large part, from the weight of the embankment fill adjacent to each abutment.

We can see only two methods for avoiding this severe differential movement. One is to keep the approach fill at least 20 feet back from the abutments and to bridge the gap with some simple approach span. As stated in a foregoing sentence, the differential settlement between abutment and pier with this arrangement is 1/270 of the span. The footing for this approach span could be founded below river bed level at approximate elevation 258 feet. The safe bearing value at this level is 3700 p.s.f. Since this footing will settle with the fill, no vertical earth pressures will be transmitted into it. The front of the footing should be protected with rip rap.

The other proposal is to carry the entire bridge on H piles, end bearing on bedrock which lies between 100 and 108 feet below river surface level. If this method of support is adopted, the H piles also will be required to carry part of the weight of the approach fill which is transferred to it by the consolidating clay. The maximum compression of this material, under the weight of the approach embankments, has been estimated to be about 7 inches. This is the value of differential movement which will develop ultimately between the clay and the tops of end bearing piles. It will diminish in more or less linear fashion to a very small value at the bottom of the compressible clay some 70 feet below river bed level. Over a major part of this length however, the differential movement should be sufficient to develop failure shear strains between the steel and the adhering clay. According to Tomlinson, this maximum adhesion force will have a value of approximately 800 p.s.f. If it is assumed, conservatively, that this force is generated over the entire 70 feet of clay, a total additional load of approximately $800 \times 4 \times 70/2000 = 112$ tons will be transferred into the lower portions of each pile. Although the underlying silty sand should be dense enough to support the piles against buckling, very high compressive stresses still will be carried by the steel and by the underlying bedrock. For this reason, it may be desirable to require the use of heavy H pile sections for end bearing support. Since the bearing value of confined rock is much higher than is indicated by tests on unconfined samples, there should be no danger of the heavily loaded piling penetrating into bedrock*.

* Pile Foundations - Chellis, p. 188

With abutments, and presumably piers as well, supported directly on bedrock, very long term differential movement or subsidence of the adjoining approach fill should be expected. Since this movement will take place very gradually over a number of years, occasional adjustments of road surface level may suffice to overcome the abrupt change of grade at the bridge entrances.

We hope that the foregoing comments assist you in deciding upon appropriate foundations for this structure.

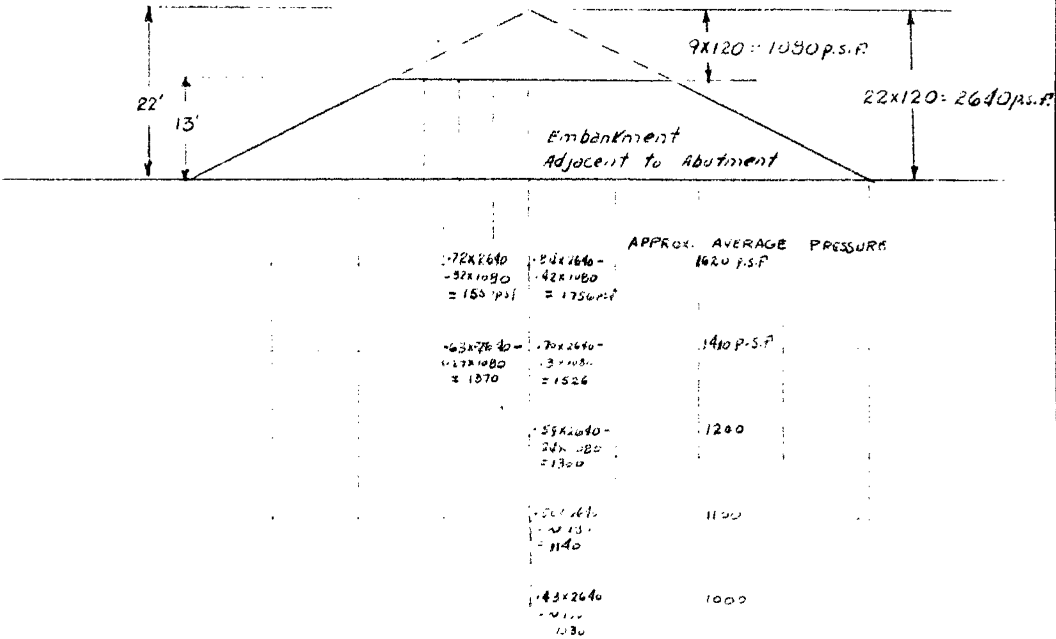
We are enclosing one copy of your preliminary bridge plan as requested.

Yours very truly,

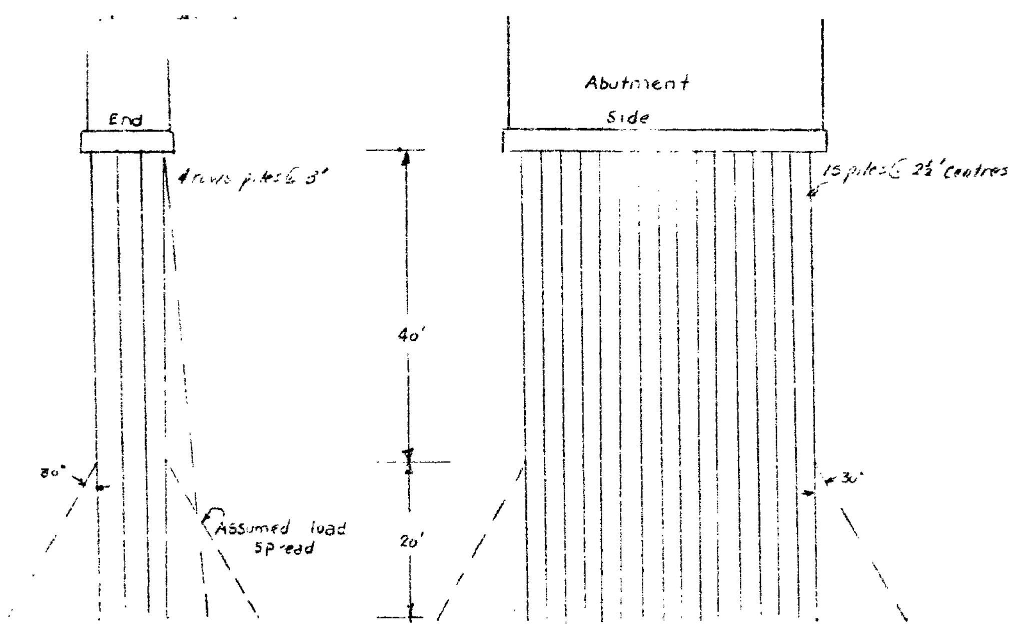
W A Trow

William A. Trow (P. Eng.)

WAT/lt
Encl.



SKETCH SHOWING ESTIMATED PRESSURE DISTRIBUTION BELOW ABUTMENTS
(Theories of Elasticity and Plasticity - Boston Soc. of Civil Engineers. 1925-1940, P.178)



SKETCH SHOWING PILE ARRANGEMENT AND ASSUMED LOAD SPREAD UNDER ABUTMENTS

SUMMARY OF SETTLEMENT COMPUTATIONS

ABUTMENTS

Scheme 1: Assume: Earth pressure for triangular loading condition as shown in Dwg. 1.
60 piles founded at Elev.205 ft., or about 60 ft. below river bed level.
4 rows of 15 piles, $2\frac{1}{2}$ ft. centres, with rows 3 ft. apart.
Approx. dimensions of pile group = 10 ft. wide by 39 ft. long.
Settlement, from abutment load, occurs below lower third of pile group, or below El. 225 ft.
30° load spread from pile group into underlying soil.

Settlement - Elev.265 - 245 ft.

P_o Elev. 255 ft. = 500 p.s.f.
e_o = 0.862 (consolid. curve for hole 1, 32 ft.)
Δp due to fill = 1600 p.s.f.
P_1 = 2100 p.s.f.
void ratio e_1 corresponding to P_1 = .82
Δe = 0.042

$$\text{Settlement } S = H\left(\frac{\Delta e}{1+e_o}\right) = 240\left(\frac{0.042}{1.86}\right) = 5.42 \text{ ins.}$$

Settlement - Elev.245 - 225 ft.

P_o Elev. 235 ft. = 1500 p.s.f.
e_o = 0.839 (32 ft. test)
Δp due to fill = 1300 p.s.f.
P_1 = 2800 p.s.f.
e_1 = .804
Δe = .035

$$S = 240\left(\frac{.035}{1.84}\right) = 4.57 \text{ ins.}$$

Settlement - Elev. 225 - 205 ft.

P_o Elev. 215 ft. = 2500 p.s.f.
e_o = 1.51 (48 ft. consolid.test)
Δp due to fill = 1100 p.s.f.

$$\Delta p \text{ due to abutment} = \frac{1200000}{(10+11.6)(39+11.6)} = 1100 \text{ p.s.f.}$$

Total p = 2200 p.s.f.
P_1 = 4700 p.s.f.
e_1 = 1.448
Δe = .062

$$S = 240\left(\frac{.062}{2.5}\right) = 5.95 \text{ ins.}$$

Settlement Computations

Abutments - Scheme 1 (Cont)

Settlement Elev. 205 - 185 ft.

P_o Elev. 195 ft. = 3500 p.s.f.
e_o = 1.28 (64 ft. consolid. test)
Δp due to fill = 1000 p.s.f.
Δp due to abutment = 364 p.s.f.
Total Δp = 1364 p.s.f.
P_1 = 4864 p.s.f.
e_1 = 1.232
Δe = .048

$$S = 240 \left(\frac{.048}{2.3}\right) = 5.00 \text{ ins.}$$

Total Settlement = 5.42 + 4.57 + 5.95 + 5.00 = 20.94 ins.

* Adjust for overconsolidation = 20.94 x .6 = 12.6 ins.

NOTE: Actual settlement of piles due to abutment weight and fill pressure below Elev. 225 feet = 11 ins., or about 50% of this movement. The compression of the clay above Elev. 225 ft. has been added to this because any tendency of the upper clay to settle more than the piles will cause additional load to be thrown into the pile group. This in turn will cause the pile group to settle more. The computations for this condition have not been included.

Scheme 2: Assume: Conditions of Scheme 1 except use piles 20 ft. long, or to Elev. 245 ft.
Settlement resulting from abutment load occurs below Elev. 252 ft.

Settlement Elev. 265 - 252 ft. = 156 ins.

P_o at average depth 258.5 ft. = 325 p.s.f.
e_o at elev. 258.5 ft. (consol. curve for 32 ft. hole 1) = .868
Δp due to weight of fill = 1700 p.s.f.
P_1 = 2025 p.s.f.
e_1 = .822
Δe = .046

$$S = 156 \left(\frac{.046}{1.87}\right) = 3.84 \text{ ins.}$$

* Settlement Analysis of Foundation Clay-
Skempton & Bjerrum Fig. 1: Geotechnique Dec. 1957

Settlement Computations

Abutments - Scheme 2 (Cont.)

Settlement Elev. 252 - 232 ft. = 240 ins.

P_o at Elev. 242 ft. = 1150 p.s.f.
e_o (curve 32 ft. hole 1) = .848
Δp due to fill = 1400 p.s.f.

Δp due to abutment = $\frac{600 \text{ tons}}{(10+11.6)(39+11.6)}$ = 1100 p.s.f.

Total Δp = 2500 p.s.f.
P_1 = 3750 p.s.f.
e_1 = 0.788
Δe = 0.060

$S = 240 \left(\frac{.060}{1.85}\right) = 7.8$ ins.

Settlement Elev. 232 - 212 ft.

P_o at Elev. 222 = 2150 p.s.f.
e_o (consolid.curve 48 ft. hole 1) = 1.519
Δp due to fill = 1100 p.s.f.
Δp due to abutment = $\frac{600 \text{ tons}}{(10+34.8)(39+34.8)}$ = 364 p.s.f.

total Δp = 1464 p.s.f.
P_1 = 3614 p.s.f.
e_1 = 1.484
Δe = .035

$S = 240 \left(\frac{.035}{2.5}\right) = 3.36$ ins.

Settlement Elev. 212 - 192 ft.

P_o at Elev. 202 ft. = 3150 p.s.f.
e_o (consolid. curve 64 ft. hole 1) = 1.29
Δp from fill 1000 p.s.f.
Δp from abutment $\frac{600 \text{ tons}}{(10+58)(39+58)}$ = 182 p.s.f.

Total Δp = 1182 p.s.f.
P_1 = 4332 p.s.f.
e_1 = 1.250
Δe = .04

$S = 240 \left(\frac{.04}{2.29}\right) = 4.19$ ins.

Total Settlement = 3.84 + 7.80 + 3.36 + 4.19 = 19.2 ins.

Adjust for overconsolidation = 19.2 x .6 = 11.5 ins.

Settlement Computations

Abutments - Scheme 3

Move fill 20 feet back from abutments and use simple approach slab shown in Dwg. 2.

Use 20 foot piles; use 30 piles, $2\frac{1}{2}$ ft. centres, placed in two rows $3\frac{1}{2}$ feet apart, - piles driven to Elev. 245 ft.

Area of pile group = $39 \times 4\frac{1}{2}$ = 175 sq.ft.

Ultimate Capacity of 30 piles = $9CA + 2(b+L)d \times C_a$

where A = 175 sq.ft.
L = 39 ft.
b = $4\frac{1}{2}$ ft.
d = depth of piles = 20 ft.
C = shear strength of soil below pile tips = approx. 1500 psf.
C_a = adhesion around pile group perimeter = approx. 800 psf.*

Ult. Cap. = 1180 + 696 = 1876 tons.
Actual Pile load = 600 Tons
Factor of Safety = 3.25

Settlement Elev. 265 - 252 = 13 ft. = zero (no fill pressure)

Settlement will occur below lower third of pile group = Elev. 252 ft.

Settlement - Elev. 252 - 232 ft.

P_o Elev. 242 ft. = 1150 p.s.f.
e_o = 0.85 (curve 32 feet)
Δp from fill = 200 p.s.f. (assume pressure distribution Case A, Plane C, p.175 of Theories of Elasticity and Plasticity - Cont. to Soil Mechanics, Boston Soc. of Civil Engineers, 1925-1940).

$$\Delta p \text{ from abutment} = \frac{600 \text{ Tons}}{(39+11.6)(4.5+11.6)} = 1475 \text{ p.s.f.}$$

Total Δp = 1675 p.s.f.
P_1 = 2825 p.s.f.
e_1 = .803

$$S = 240\left(\frac{.047}{1.85}\right) = 6.1 \text{ ins.}$$

* Adhesion of Piles driven in Clay Soils - M. J. Tomlinson: Proceedings 4th Int. Conf. Soil Mechanics & Foundation Engineering 1957.

Settlement Computations

Abutments
Scheme 3 (cont.)

Settlement Elev. 232 - 212 ft.

P_o - Elev. 222 ft. = 2150 p.s.f.
e_o = 1.519 (curve 48 ft.)
Δp embankment = 430 p.s.f.
Δp abutment $= \dfrac{700 \text{ tons}}{(39+34.8)(4.5+34.8)} = 414$ p.s.f.

Total Δp = 844 p.s.f.
P_1 = 2994
e_1 = 1.50
Δe = .019

$S = 240 \left(\dfrac{.019}{2.5}\right) = 1.83$ ins.

Settlement Elev. 212 - 192 Ft.

P_o = 3150 p.s.f.
e_o = 1.29 (curve 64 ft.)
Δp embankment = 450 p.s.f.
Δp abutment = 198 p.s.f.
Total Δp = 648 p.s.f.
P_1 = 3800
e_1 = 1.27

$S = 240 \left(\dfrac{.02}{2.29}\right) = 2.1$ ins.

Total settlement = 6.1 + 1.8 + 2.1 = 10.0 ins.

Allowing for over consolidation = 10 x .6 = 6 ins.

Estimated adjusted settlement of piers = 3.6 ins.

Total Differential = 2.4 ins. $= \dfrac{2.4}{645} = \dfrac{1}{270}$ of span

Since this differential movement will occur at a very slow rate, concrete members should be able to accommodate the resulting redistribution of stress.

ESTIMATED PIER SETTLEMENT

Found piles at Elev. 235 feet, or 22 feet below river bed level.

Use 2 rows of 12 piles at 3 ft. centres, (for check on ultimate capacity see computation, Scheme 3, for abutments).

Dimensions of pile group = $4\frac{1}{2}$ by 37 feet.

Lower third of pile at Elev. 242 feet.

Consider Settlement Elev. 242 - 222 feet

P_o Elev. 232 = 1250 p.s.f.

e_o = .847 (curve 32 feet)

$$\Delta p = \frac{425 \text{ tons}}{(37+11.6)(4.5+11.6)} = 1085 \text{ p.s.f.}$$

P_1 = 2335 p.s.f.

e_1 = .817

Δe = ,04

$$S = 240 \left(\frac{.04}{1.85}\right) = 5.2 \text{ ins.}$$

Consider Settlement Elev. 222 - 202 ft.

P_o Elev. 212 ft. = 2250 p.s.f.

e_o = 1.515 (curve 48 ft.)

$$\Delta p = \frac{425 \text{ tons}}{(37+34.8)(4.5+34.8)} = 300 \text{ p.s.f.}$$

P_1 = 2550 p.s.f.

e_1 = 1.51

Δe = .005

$$S = 240 \left(\frac{.005}{2.5}\right) = 0.5 \text{ ins.}$$

Estimated total settlement = 6 ins.

Adjustment for over consolidation = 6 x .6 = 3.6 ins.

ESTIMATED SETTLEMENT OF EMBANKMENT

Assume adjusted terrace loading shown in Dwg. 1

Settlement 265 - 252 ft. - 3.84 ins. (See Scheme 1)

Settlement 252 - 232 (See Scheme 1)

P_o = 1150
e_o = .848
Δp = 1400
P_1 = 2550
e_1 = .808
Δe = .040

$S = 240 \left(\frac{.04}{1.85}\right) = 5.2$ ins.

Settlement 232 - 212 ft. (See Scheme 1)

P_o = 2150 p.s.f.
e_o = 1.519
Δp = 1100 p.s.f.
P_1 = 3250 p.s.f.
e_1 = 1.493
Δe = .026

$S = 240 \left(\frac{.026}{2.5}\right) = 2.5$ ins.

Settlement 212 - 192 ft. (See Scheme 1)

P_o = 3150 p.s.f.
e_o = 1.29
Δp = 1000
P_1 = 4250
e_1 = 1.255
Δe = .034

$S = 240 \left(\frac{.034}{2.29}\right) = 3.56$ ins.

Total settlement = 11.26

Adjustment for over consolidation = 11.3 x .6 = 6.8 ins.

WILLIAM A. TROW AND ASSOCIATES

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

W. A. TROW, M.A.S.C., M.E.I.C., P.ENG.

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

Project: J343

March 16, 1959.

59-F-239

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

Attention: Mr. J. McAllister

Gananoque River Crossing
Hwy. No.32 - Proposed Revision Line C

Dear Sirs:

Enclosed herewith is our report on the soil conditions existing at the above-noted bridge site.

This site was found to be underlain by a very deep deposit of plastic clay which exists in a stiff overconsolidated state at upper levels. Bedrock lies about 110 feet below river surface level.

In view of the stiff nature of the upper levels of clay and the fact that the river flow is rather sluggish, a floating pile foundation scheme seems to have considerable merit for this bridge structure. An analysis, based on certain assumptions regarding pier and abutment loads, has been made for a proposal using short timber piles bearing 30 feet below river bed level. A long term differential settlement between abutments and piers of 5 inches has been computed. The greater settlement was at the abutment locations and results from the application of 10 feet of approach fill. Because of this fact, there appears to be merit in using more heavily loaded centre piers in order to increase the settlement in those parts of the structure. The use of one centre pier instead of two may provide this extra weight.

The alternative method of support, of course, is the use of end-bearing H piles resting directly on bedrock. Although no settlement would result from this arrangement, the cost of foundations would be considerably greater. In addition, since the adjacent fill approaches will continue to settle slowly for several years, occasional adjustments of grade will be necessary in order to avoid a bump at each end of the bridge. End-bearing piles must be designed to carry a portion of this fill weight.

If, after your review of the contents of this report, you feel that the floating pile scheme has some merit, we shall be pleased to review our settlement analysis inserting values that may be more appropriate.

It has been our pleasure, again, 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.

GANANOQUE RIVER CROSSING
HWY. NO. 32, PROPOSED REVISION LINE C

Project: 343

March 16, 1959.

William A. Trow and Associates

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ENCLOSURES

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GANANOQUE RIVER CROSSING
HWY.NO. 32 - PROPOSED REVISION LINE C

This report presents the results of an investigation consisting of four borings and eight penetration tests completed during February, in order to determine the foundation conditions underlying this proposed river crossing. Two foundation proposals for bridge support have been discussed.

Description of Site

The site of the proposed river crossing represents a slight local revision to the location of Hwy. 32, and is about 200 feet east, or upstream, of the existing Bailey bridge structure. The river is quite slow moving at this point and little or no signs of erosion are in evidence.

Bedrock outcrops at several locations just east and west of the river and at points upstream and downstream of the proposed crossing. During flood periods, the river spills over its flat banks up to this rock which rises about 10 to 20 feet above the surrounding terrain. Other surface features were somewhat obscured by heavy snow cover.

Scope of Field Investigation

The locations of the four borings and eight penetration tests of this investigation are shown in Dwg. 1. This represents a slight change in the original investigation and was done at the request of Mr. Lock of the Department of Highways Bridge Office, who visited the site in the early stages of the work.

The testing was begun on the north side of the river by driving a 2-inch diameter cone at the borehole No.1 location. In this dynamic penetration test, a 2-inch cone is driven from the surface downward under an energy of 350 ft. lbs. per blow exerted by a 140 pound hammer. The number of blows required for each foot of penetration are recorded. At this location the test was terminated at a depth of 90 feet because of a temporary lack of drill rods. This situation was quickly remedied and all remaining borings and penetration tests were taken to refusal.

After this test was completed, the drill was moved about 5 feet and the process of soil sampling was begun. Samples were taken at five foot intervals in this first boring so that a close check on subsoil stratigraphy could be obtained. For the most part the soil was recovered in a disturbed state using a 2-inch O.D. split spoon, but some undisturbed 2-inch I.D. Shelby tube samples were recovered at representative depth intervals. All samples were sealed to prevent moisture loss. The split spoon was driven into the soil using the same energy indicated above for the cone penetration tests. The Shelby tubes were either pushed or levered into the ground.

Field vane test measurements were made at 5 foot intervals after each sample was recovered. The vane was pushed into and 18 inches below the hole left by the sample spoon. A correction was applied to all vane measurements in order to account for friction acting on the rods.

The other borings and penetration tests were performed in the same manner as indicated above except that a wider sampling interval was employed. Bedrock was proven to a depth of 5 feet in hole 2 and for 8½ feet in hole 4, with a recovery of 71% and 89% respectively. All elevation measurements were referred to ice level on the river which was established as Elev. 268.8 ft. at the time of the investigation.

Laboratory Testing

Some laboratory testing was carried out in order to make an appraisal of the strength and compressibility of the deep deposits of clay at this site. The strength determinations were made on 4 representative undisturbed samples from hole 1 and one sample from hole 3. These tests were of the undrained triaxial type performed at overburden pressure conditions. They supplemented the measurements of clay shear strength obtained by the field vane.

In order to obtain an indication of the compressibility of the clay, moisture content measurements were obtained on each sample. Seven Atterberg limit tests were carried out on material from hole 1 so that these moisture content values could be referenced to the plastic range of the clay. Three consolidation tests were performed on samples from depths of 32, 48 and 64 feet in hole 1. These tests form the basis for computations of settlement for the floating pile foundation scheme suggested later in the report. The results of these various tests are recorded in Table 1 and Dwgs. 2 to 9.

Description of Soil Types and Discussions of Test Results

The borehole logs for holes 1 to 4 are shown in Dwgs. 2 to 5; the estimated subsurface stratigraphy, indicated in Dwg. 1, is based upon the information presented in these logs. Reference to this latter drawing shows that the predominate soil type at the site is a stiff grey silty clay. Except for a thin film of river mud and some sand and gravel, this material extends right to river bed level. In the borehole logs, it has been divided into two strata. The upper layer consists of an unstratified deposit of stiff fissured silty clay with random pebbles and some pockets of sand. Below a depth of about 35 feet, the soil becomes faintly stratified with layers of less plastic silty clay located at irregular intervals. This stratification could be noted on samples that had been sectioned and allowed to dry.

According to field and laboratory measurements, the two layers have similar physical properties. The shear strength, according to field vane measurements ranges from 1000 to 2000 p.s.f. Some laboratory undrained triaxial test results indicated lower strengths but, in at least one instance, this was because the sample contained considerable silt which tended to slump during the preparation of the test.

The clay, generally, appeared to be quite plastic with a liquid limit close to 60 percent and a plastic limit of the order of 22 percent. However, local variations were visually noted or could be inferred by the variations in the moisture content measurements. Truly representative measurements of moisture content and plasticity were not possible because it was difficult to separate the clay from the pockets of sand and the layers of more silty clay. The lower moisture contents and Atterberg limits noted at depths of 35 to 40 feet indicate a much less plastic condition at these levels. This could represent the transition zone between the upper and lower deposits of clay.

Of particular significance with regard to bridge foundations at this site is the fact that the clay above a depth of about 40 feet appears to exist in an over-consolidated state. This overconsolidation is indicated by the results of the consolidation test from a depth of 32 feet in hole 1, by the high shear strength measurements obtained by the field vane, and by the position of the moisture contents within the plastic range at these upper levels.

Overconsolidation is also suggested by the consolidation test result for the 48 foot depth and by the high field vane measurements at this level. The very compressible condition indicated by this consolidation measurement was thought to be the result of a test failure due to plastic flow of the clay sample under higher load increments. However, no visible evidence of clay displacement was noted upon completion of this test.

Attention has been drawn to the overconsolidated state of the upper levels of clay because advantage can be taken of this condition in the support of the bridge. An analysis of a pile foundation scheme, floating in this stiff material, has been given in the Appendix.

At a depth of 64 feet, the clay shows little evidence of overconsolidation. The estimated preconsolidation pressure is similar to the existing overburden pressure and the field moisture contents lie close to the liquid limit as would be expected for a clay at this depth below the surface. Therefore any addition of pressure from external sources will cause much higher settlement than would result in the overlying overconsolidated clay.

Below a depth of about 80 feet the clay stratum gradually blends into the underlying fine sandy silt. This stratified deposit contains some clay layers but is essentially granular in character.

Bedrock is located at a depth of about 105 feet. In hole 2 it was found to consist of the hard granite gneiss which outcrops in the area. However, in hole 4, metamorphosed limestone with patches of serpentine and some seams of gypsum were noted. Good recovery was obtained at both locations.

Foundation Considerations

As stated in the introductory remarks to this letter, two schemes for support of the proposed bridge have been examined.

If no settlement of consequence can be tolerated, the most positive means of support is to use end-bearing piles driven right down to bedrock. Since bedrock at the site extends to a depth of almost 110 feet below the river surface, very long piles would be required. High capacity steel H piles may be the most economic means of support under this circumstance. The stiff clay at higher levels provides sufficient support to prevent buckling of 12 inch H piles; accordingly, the maximum loads to apply will be determined by their rated capacity when considered as short columns. The measurements at seven test locations indicate that the bedrock surface is relatively flat and therefore there appears to be no danger of the pile tips sliding along this bearing surface.

An alternative scheme which could be considered is one involving a pile foundation floating in the clay deposits. Since the clay appears to have a uniform thickness across the site and its physical properties were found to be similar for any given level, at all locations, the magnitude of any long term settlement will be determined by the loading applied. Therefore, it may be possible to adjust this loading so that both piers and abutments will settle the same amount. The advantage of this scheme is the possible saving in foundation costs through the use of simple timber piles driven to much shallower depths than bedrock.

An analysis of this floating foundation proposal has been given in the Appendix. Certain assumptions regarding pier and abutment loads and dimensions have been made in order to provide some basis for the computations. The centre piers have been assumed to carry a maximum load, inclusive of concrete and gravel-fill weight, of 700 tons. The abutments loads have been estimated as 300 tons.

Two alternatives of this latter proposal have been analysed. One involves support of piers and abutments on timber piles at a depth of 30 feet below river level, or Elev. 239 feet. The other requires fewer piles but they are driven to a depth of 70 feet, or Elev. 199 feet. In the former case a long term settlement for the piers of the order of 5 inches, was computed. An estimated settlement of double this value was computed for the abutments and this was due, in very large part, to the weight of the approach fill which compresses a considerable depth of the underlying clay. The differential movement between these two reaction points is 5 inches and this applies over a span of approximately 55 feet. The duration of this differential movement has been estimated very approximately to be of the order of four decades.

Settlement of a slightly higher magnitude was computed in the analysis of the deep floating pile scheme. Therefore no advantage is derived from using fewer piles driven to this depth, particularly since the overall pile footage is greater.

In these analyses no allowance has been given for the support provided by the soil at river bed level either directly to the pile cap or to an intermediate stone-filled crib. If precautions are taken to displace the river mud and to ensure that the stone or gravel in the crib is consolidated into a dense state before placing the pile cap, the support at river bed level should be considerable. The only factors that may reduce this support are river bed scour and the disturbance and heave of the soil during pile driving operations. River scour does not seem to be an important item at this site; the magnitude of ground heave will diminish with the wider pile spacing. In addition it should be appreciated that the compressibility of natural deposits of clay is usually less than indicated by laboratory test particularly when recompression is involved as is the case in this instance. Finally, it is probable that the abutment and pier loads will be less than assumed in this analysis. This, unfortunately, will result in greater differential settlement because the embankment fill weight will not differ. In view of the effect of this fill which produces greater settlements at the abutments, it may be desirable to increase the loading on the piers in order to obtain equivalent movement at these locations. This could be done by revising the bridge design from a three span to a two span proposal.

Conclusions

The comments of the foregoing sections can be summarized briefly as follows:

1) The bridge site is underlain by a deep deposit of very plastic clay which is dessicated or overconsolidated for the first approximately 40 feet. Bedrock lies almost 110 feet below the river surface and it consists of the granite gneiss which outcrops north and south of the proposed crossing. Metamorphosed limestone was encountered in the vicinity of the south abutment however. Between this bedrock level and a depth of 80 feet, the approximate bottom surface of the clay, is a deposit of medium dense sandy silt with some clay layers.

2) Two proposals for pile support have been considered. One involves support on H piles bearing directly on bedrock; the other incorporates the use of short wood piles bearing some 30 feet below river surface level. The former method is to be preferred if no settlement can be tolerated, but the cost of the piles will be considerably greater. In the latter proposal, a total settlement of 5 and 10 inches has been estimated for the piers and abutments respectively. This movement should continue over a period of 40 years. The settlement estimates are based upon certain assumptions regarding bridge loads. Different movements can be expected if these loadings do not apply. In any event, in view of the considerable consolidating effect of the approach fill, it may be wise to increase the loading on the centre piers in order to minimize differential movement.

(3) No embankment stability problem exists at this site and the underlying clay should remain stable during pile driving operations.

REGISTERED PROFESSIONAL ENGINEER
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PROVINCE OF ONTARIO

W Trow

William A. Trow (P. Eng.)

WAT/lt
March 16, 1959.
J343

APPENDIX

ANALYSIS OF FLOATING PILE FOUNDATION SCHEME

In order to provide some basis for analysis, the assumption is made that the abutment loads are 300 tons each and the centre pier loads, inclusive of concrete and stone-filled crib, are 700 tons each.

A - Centre Piers

1. Support on Short Piles driven to Elev. 239 ft.

(a) Ultimate Capacity:

Piles in this area extend 20 feet below the river bed.

Ultimate capacity per 1 ft. diameter pile =

$$9CA + Phs = \frac{9 \times 1200 \times .78 + \pi \times 20 \times 800}{2000} = 29 \text{ tons}$$

where C = 1200 is the value of cohesive resistance of the clay below the pile tips, believed to be appropriate.
A is the pile cross section area.
P is the pile perimeter
h = 20 feet is the penetration depth of the pile
s is the available skin friction on the pile shaft*

Use 3 rows of 16 piles each at $2\frac{1}{2}$ foot centres under a pile cap 8 feet wide by 40 feet long.

Load per pile $\frac{700}{48} = 14.6$ tons.

Factor of safety $= \frac{29}{14.6} = 2.0$

Ultimate Capacity of Pile Group = 9CA + Phs =

$$\frac{9 \times 1200 \times (8\times40) + 2(8+40) \times 20 \times 800}{2000} = 2500 \text{ tons}$$

Factor of safety $= \frac{2500}{700}$ 3.6

* The Adhesion of Piles Driven in Clay Soils
M.J. Tomlinson - Proceedings 4th Int.Conf. Soil Mechanics and Foundation Engineering 1957.

(b) Settlement of Pile Group

Compressed soil lies at and below lower third of pile group; load spread at 30° to the vertical (Terzaghi, Bjerrum et al)

Layer 1: 23 - 43 feet H = 240 ins.

Average depth = 33 ft.
Existing in situ pressure Po = 23 x 50 psf = 1150 psf

Unit loading, from pier, at 33 ft. = $\frac{700}{(8+11.6)(40+11.6)}$ = .69 tsf = Δ p

S = H Δpmv = 240 x .69 x .0259 = 4.3 ins.

mv = .0259 = coefficient of compressibility from consolidation curve hole 1 - 31 ft.

Layer 2: - 43 - 53 ft. H = 120 ins.

Average depth = 48 ft.

Po = 38 x 50 = 1900 psf

Unit loading from pier at 48 ft. = $\frac{700}{(8+29)(40+29)}$ = .275 tsf

S = 120 x .275 x .0218 = 0.72 ins.

mv = .0218 obtained from consolidation curve hole 1 - 48 ft.

Total settlement ~ 5 inches.

This value believed to be high because actual in situ value of mv should be smaller and in addition, part of pier load will be carried at river bed level by the gravel crib.

2. Support on Long Piles driven to Elev. 199 ft.

(a) Ultimate capacity of pile =

$$\frac{9 \times 1200 \times .78 + \pi \times 60 \times 800}{2000} = 79 \text{ tons}$$

Use 3 rows of 10 piles at 4 foot centres - 30 piles

Load per pile = $\frac{700}{30}$ = 23.3 tons

Factor of safety $\sim$ 3

Similarly, factor of safety, pile group $>$ 4.

(b) Settlement of Deep pile Group

Compressed soil at and below 50 feet (lower third of pile group)

Layer 1: 50 - 70 ft. H = 240 inches.

Average depth = 60 ft.

Po below stream bed = 50 x 50 = 2500 psf.

Unit loading at 60 ft. = $\frac{700}{(8+11.6)(40+11.6)}$ = .69 tsf.

S = 240 x .69 x .03 = 5 ins.

mv = .03, from consolidation curve, hole 1 - 64 ft.

Layer 2: 70 - 80 ft. H = 120 ins.

Average depth = 75 ft.

Po = 65 x 50 = 3250 psf.

Unit loading = .275 tsf.

S = 120 x .275 x .03 = 1.0 ins.

mv $\sim$ .03 from consolidation curve, hole 1 - 64 ft.

Total settlement = 6 ins.

B. Abutments

1. Settlement due to weight of approach fill

Assume fill 30 ft. wide at top and 10 ft. deep.
Consider layer from ground surface to depth of 40 ft.
Average depth = 20 ft.

Po at 20 ft. = 20 x 50 = 1000 psf

P due to fill 0.85 x 10 x 120 = 1020 psf = .51 tsf

mv from consolidation curve, hole 1 - 32 ft. = .03 sq.ft./ton

Therefore: S = 12 x 40 x .51 x .03 = 7.34 ins.

A small amount of deep seated settlement should be expected below this level.

2. Support of Piles at Elev. 239 ft.

Assume abutment load = 300 tons.
Ultimate capacity slightly greater than for centre piers.
Use 15 tons per pile; number of piles $\frac{300}{15} = 20$

Use 2 rows of 10, placed at 4 foot centres.
Approximate dimensions of pile cap = 40 by 6 feet.

Settlement considerations:

Lower third point of piles begins at depth of approx. 21 ft.
Consider layer 21 - 41 ft. H = 20 x 12 = 240 ins.

Po at mid depth of 31 ft. = 50 x 31 = 1550 psf.

Unit loading from piles at 31 ft. for 30° load spread =

$$\frac{300}{(6+11.6)(40+11.6)} = 0.33 \text{ tsf.}$$

mv from consolidation curve 32 ft. = .0273 sq.ft./ton

S = 240 x .33 x .0273 = 2.16 ins.

Add ½ inch for deep seated settlement = 2.7 ins.

3. Support of Piles at Elev. 199 ft.

Ultimate capacity per pile = $\frac{9 \times 1200 \times .78 + \pi \times 70 \times 800}{2000} = 92$ tons

Use 2 rows of 8 piles placed at $5\frac{1}{2}$ foot centre, with rows 4 ft. apart.
Load per pile = $\frac{300}{16} \sim 19$ tons.

Settlement considerations:

Lower third point of pile group begins at depth of 46 feet.
Consider layer 46 - 76 feet: $H = 30 \times 12 = 360$ ins.
Po at mid depth of 61 feet = $61 \times 50 = 3150$ psf.

Unit loading at 61 feet = $\frac{300}{(6+17.4)(40+17.4)} = 0.223$ tsf.

mv from consolidation curve for 64 feet = 0.039 sq.ft/ton

$S = 360 \times .223 \times .039 = 3.1$ ins.

C. Summary

Settlement of Centre piers at Elev. 239 ft. = 5 ins.
Total pile footage = $20 \times 48 = 960$ ft.

Settlement of Centre piers at Elev. 199 ft. = 6 ins.
Total pile footage = $60 \times 30 = 1800$ ft.

Settlement of abutments including effect of approach fill:

Founded at Elev. 239 ft. - $\sim$ 10 ins.
Total pile footage = $20 \times 30 = 600$ ft.

Founded at Elev. 199 ft. - $\sim$ $10\frac{1}{2}$ ins.
Total pile footage = $16 \times 70 = 1120$ ft.

More economic proposal - wood piles founded at Elev. 239 ft.

Differential settlement from abutment to centre pier - $5\frac{1}{2}$ ins.
Ratio of deflection to span = $\frac{5}{55 \times 12} = \frac{1}{132}$

Duration of settlement, taking coefficient of consolidation Cv very approx. = .06 sq.ft./dqy and thickness of drainage layer = 30 ft.:

Fifty percent settlement $\sim$ 9 yrs.
Ninety percent settlement $\sim$ 40 yrs.

Settlement estimates believed to be high because of support at pile cap level, assumed loads may be too high, and compressibility of soil less than laboratory tests indicate.

D. Alternative Foundation Proposal

Use steel H piles to bedrock.

Assume 60 tons per pile.

No. of piles, centre piers $= \frac{700}{60} = 12$

Footage = 12 x 100 = 1200 feet.

Abutments $= \frac{300}{60} = 5$

Footage = 5 x 110 = 550 ft.

Settlement - zero.

TABLE NO. 1

SUMMARY OF FIELD AND LABORATORY TESTS

Depth	Shear Str.K.S.F. & Pene.Resis. N				Nat. Moist. Cont. % Dry Weight				Nat. Unit Weight pcf			
	Hole No.				Hole No.				Hole No.			
Feet	1	2	3	4	1	2	3	4	1	2	3	4
10	N=7 V=>2.2			N=9	24.8			20.1				
15	N=7	N=push V=1.34	N=7 V>2.2		21.2	37.2	39.9					
20	N=9 Qu=1.55			N=9	48.7 LL=63.9 PL=26.7			37.6	111.4			
25	N=7 V=1.6	N=push	N=7		41.4 LL=62.3 PL=21.5	48.1	37.6					
30	N=5 Qu=0.72 V=1.35	V=.93	V=2.06	N=5	32.6 LL=44.4 PL=19.4			30.8	110.8			
				Check 31 ft. H 1	MC=28.6 LL=40.4 PL=17.9							
35	N=push			V=1.52	28.6 LL=29.0 PL=15.5							
		N=push	N=push			27.4	24.2					
40	V>2.2	V=1.6	V=1.68									
45	N=11			N=4 V=1.76	44.5			52.8				
	N=4 Qu=0.8	N=push	N=push		57.5 LL=60.6 PL=24.0	47	55.4		105.6			
50	V=1.68	V=1.34	V=1.51									
55	N=push V=1.51			N=3 V=1.52	57.0			52.9				
	N=push	N=push	N=push Qu=0.63		47.0		47.4					108.1

TABLE NO.1 (CONT)

Depth	Shear Str.K.S.F. & Pene.Resis. N Hole No. 1	2	3	4	Nat. Moist. Cont. % Dry Weight Hole No. 1	2	3	4	Nat. Unit Weight pcf Hole No. 1	2	3	4
60	V=1.43	V=1.18	V=1.34									
	Qu=1.02				53.0				110.1			
	N=push			N=push	LL=60.5			37.9				
					PL=22.4							
65	V=1.09											
	N=push	N=push			39.3	34.3						
70	V=1.26	V=1.72	N=push				36.8					
			V=1.51	N=push								
75												
	N=push	N=push										
80		V=2.1	N=push									
	V=1.51											

LEGEND: V = field vane
Qu= undrained triaxial
N = penetration resistance, Blows per foot, 350 ft. lbs. energy
LL= Liquid limit
PL= Plastic limit
Mc= Moisture content

WILLIAM A. TROW AND ASSOCIATES

PROJECT NO. N 343

DRAWING NO. 2

WILLIAM A. TROW AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

LEGEND
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
CASING

Proposed Bridge Site
Gananoque River, Highway #32
See plan.
268.8

1
KP
EA
KP

River Surface 268.8
3' water

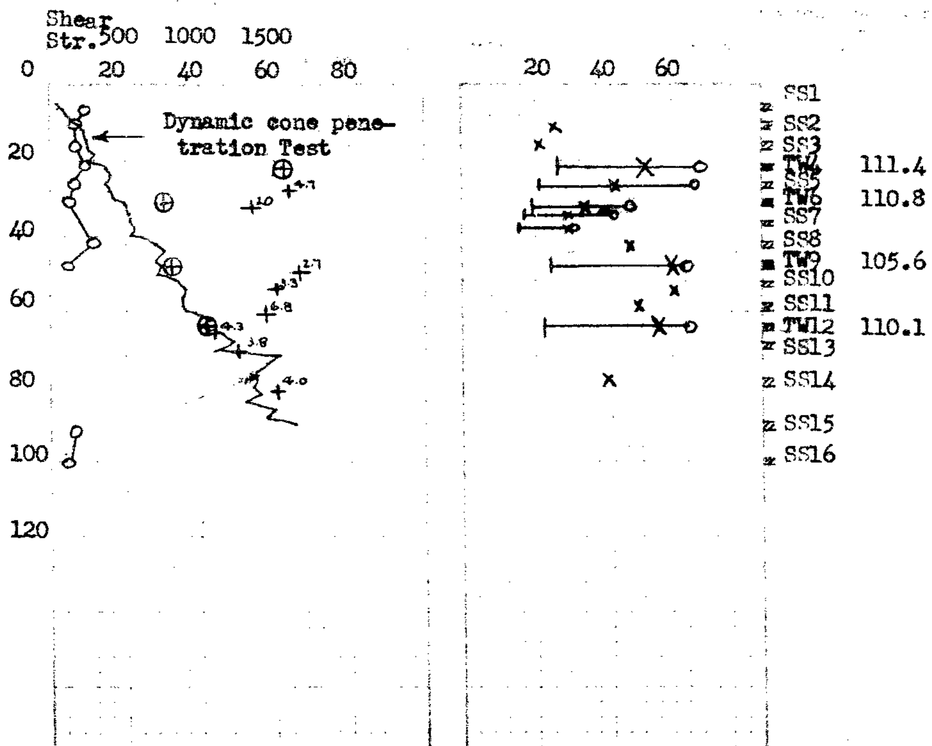
Grey stiff fissured silty clay with some pockets of fine sand and occasional stone 236

Grey stiff to stiff faintly varved silty clay, some fine sand noted below 78 ft. 189

Grey layered fine sandy silt, with small amount of clay

End of hole 161.3

Bedrock assumed.



NOTE: 1) All samples obtained by pushing sampler except where noted.
2) Approx. 6 ins. soft mud in the river bed. Some sand silt and pieces of wood noted to 6½ ft.

PROJECT NO. J 343

DRAWING NO. 3

WILLIAM A. TROW AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

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

PROJECT Proposed Bridge Site
LOCATION Gananoque River Highway #32
HOLE LOCATION See plan
268.8

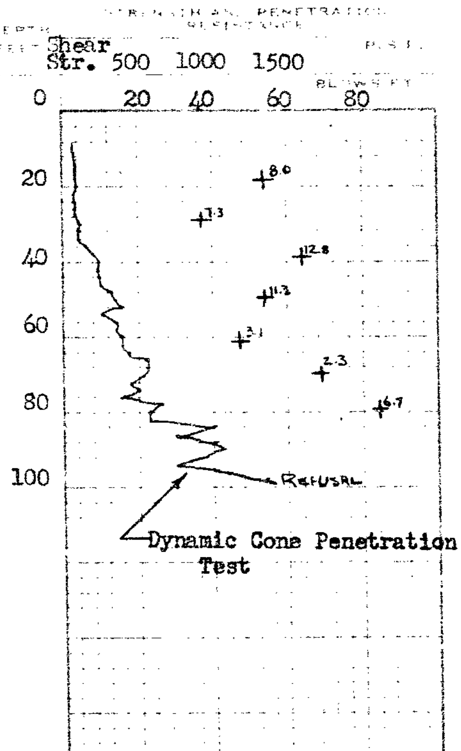
BOREHOLE NO. 2
FIELD SUPERVISOR KP
DRILLER EA
PREP KP

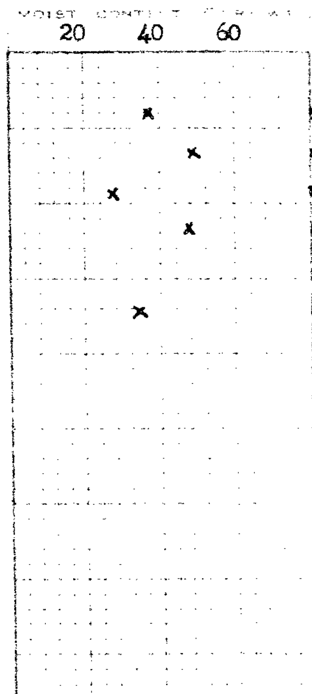
DESCRIPTION	ELEV. FEET
River surface	268.8
10½ ft. water	
River bottom	258.3
Grey stiff fissured silty clay Some fine sand & occasional stone	232
Grey stiff varved silty clay	195
Grey layered fine sandy silt with a small amount of clay	167.3
Granite gneiss	162.5

End of hole

Core AX 101.6 to 106.3
Recovery = 70%

NOTE: All split spoon samples obtained by pushing sampler into soil.





SS1
SS2
SS3
SS4
SS5
SS6
SS7
SS8

PROJECT NO. J 343

DRAWING NO. 4

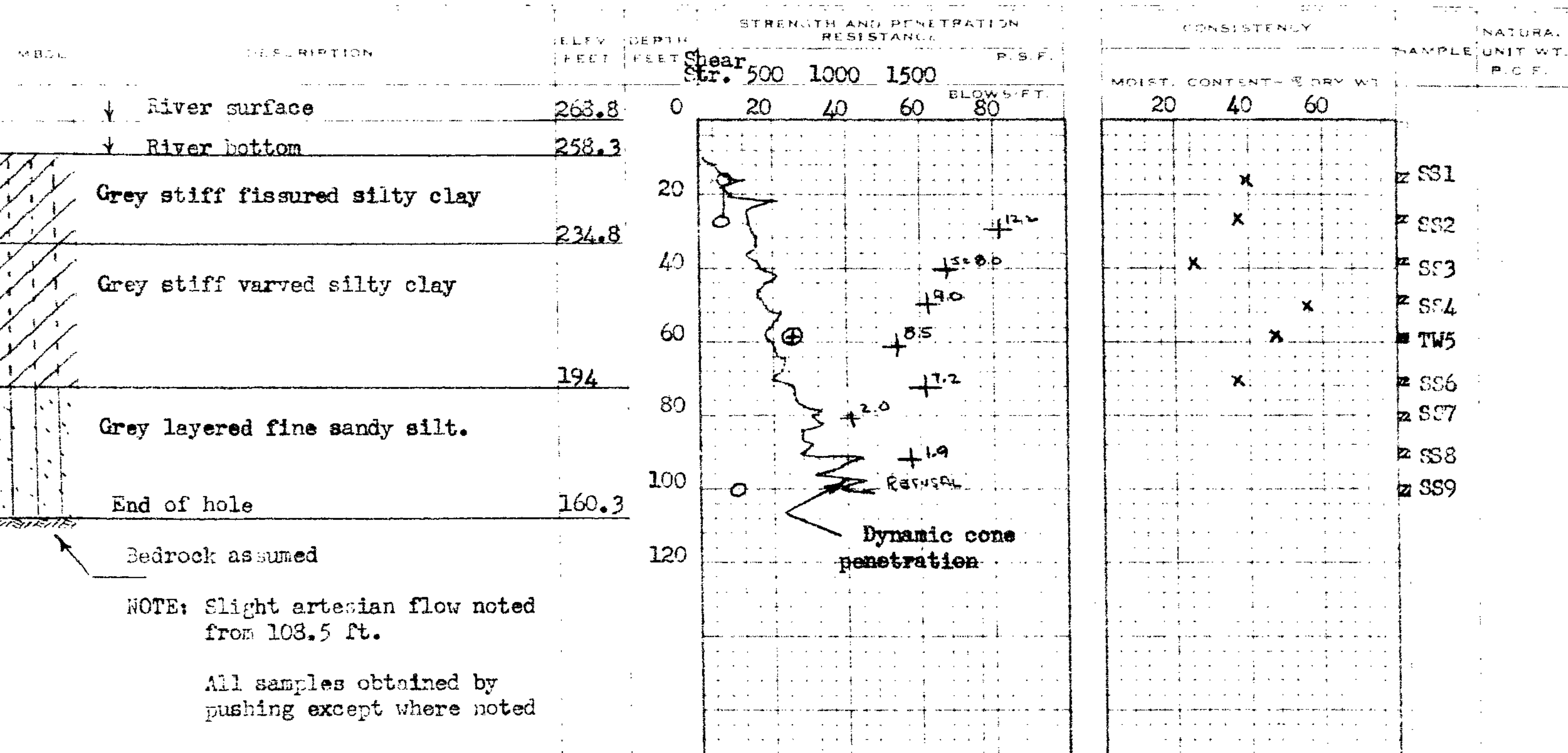
WILLIAM A. TROW AND ASSOCIATES

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 Proposed Bridge Site
LOCATION Gananoque River Hwy.#32
HOLE LOCATION See plan
ELEVATION DATUM 268.8

BOREHOLE NO. 3
FIELD SUPERVISOR MS
DRILLER EQ
PREP. KP



NOTE: Slight artesian flow noted from 108.5 ft.

All samples obtained by pushing except where noted

PROJECT NO. J 343

DRAWING NO. 5

WILLIAM A. TROW AND ASSOCIATES

INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

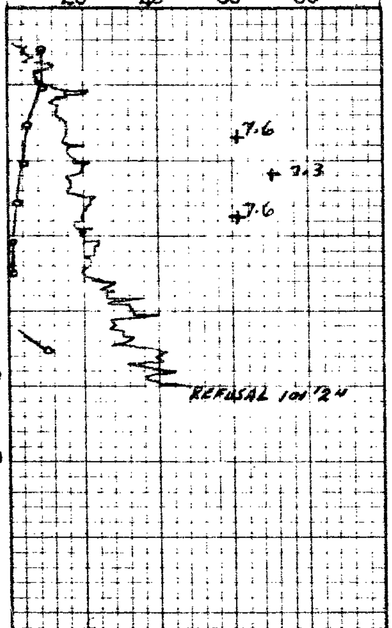
PROJECT Proposed Bridge Site
LOCATION Gananoque River Hwy.#32
HOLE LOCATION See plan
HOLE ELEVATION 268.8 DATUM

BOREHOLE NO. 4
FIELD SUPERVISOR MS
DRILLER EA
PREP. KP

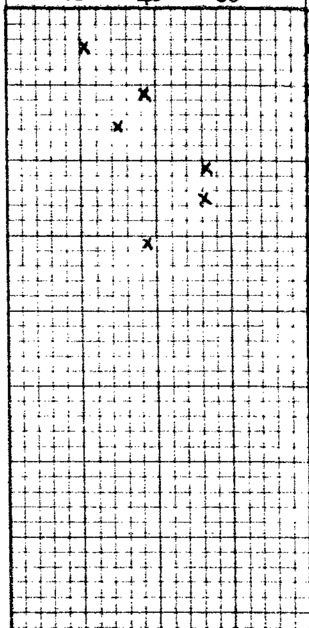
LEGEND
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
CASING
2" SHELBY
1/2 UNCONFINED COMPRESSION [Qu]
VANE TEST [C] AND SENSITIVITY [S]
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
		268.8	0
	River bed	265	
	Grey stiff fissured silty clay		20
		234	
	Grey stiff varved silty clay		40
			60
		194	
	Grey layered fine sandy silt with some clay		80
		169.3	100
	Bedrock - Metamorphosed limestone with patches of serpentine 89% recovery.		
			120
	NOTES: All samples pushed except where noted.		

STRENGTH AND PENETRATION RESISTANCE
Shear Str. 500 1000 1500 P.S.F.
BLOWS/FT. 20 40 60 80



CONSISTENCY
MOIST. CONTENT - % DRY WT. 20 40 60



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

PROJECT NO. J 343

DRAWING NO. 6

WILLIAM A. TROW 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 (Q)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

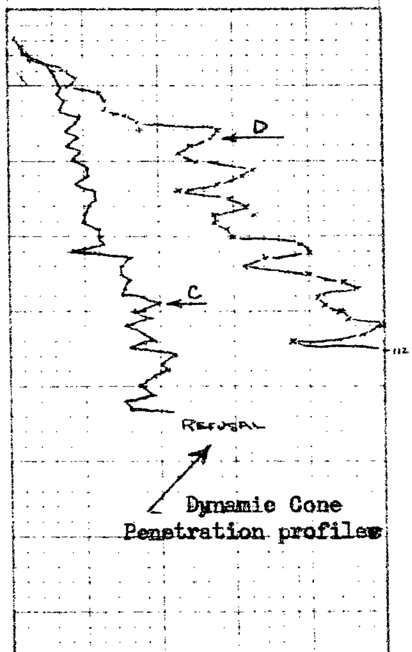
PROJECT Proposed Bridge Site
LOCATION Gananoque River Hwy.#32
HOLE LOCATION See plan
HOLE ELEVATION AND DATUM Elev.Cone C & D 268.8

BOREHOLE NO. Cone C & D
FIELD SUPERVISOR MS
DRILLER EA
PREP. KP

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	River surface	268.8	0
	Depth of Water Cone C - 10½ ft. Depth of Water Cone D - 1.5 ft. Cone D 1.5 - 7.0 ft. rods pushed		

STRENGTH AND PENETRATION RESISTANCE P.S.F.
BLOWS/FT.
20 40 60 80

Depth: 20, 40, 60, 80, 100, 120



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

PROJECT NO. J 343

DRAWING NO. 7

WILLIAM A. TROW AND ASSOCIATES

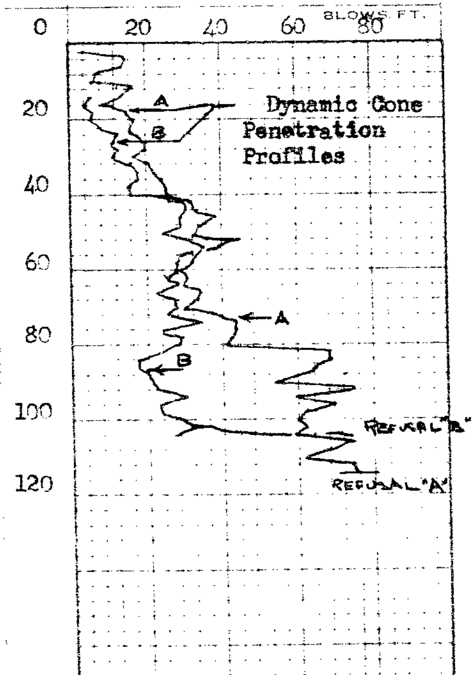
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

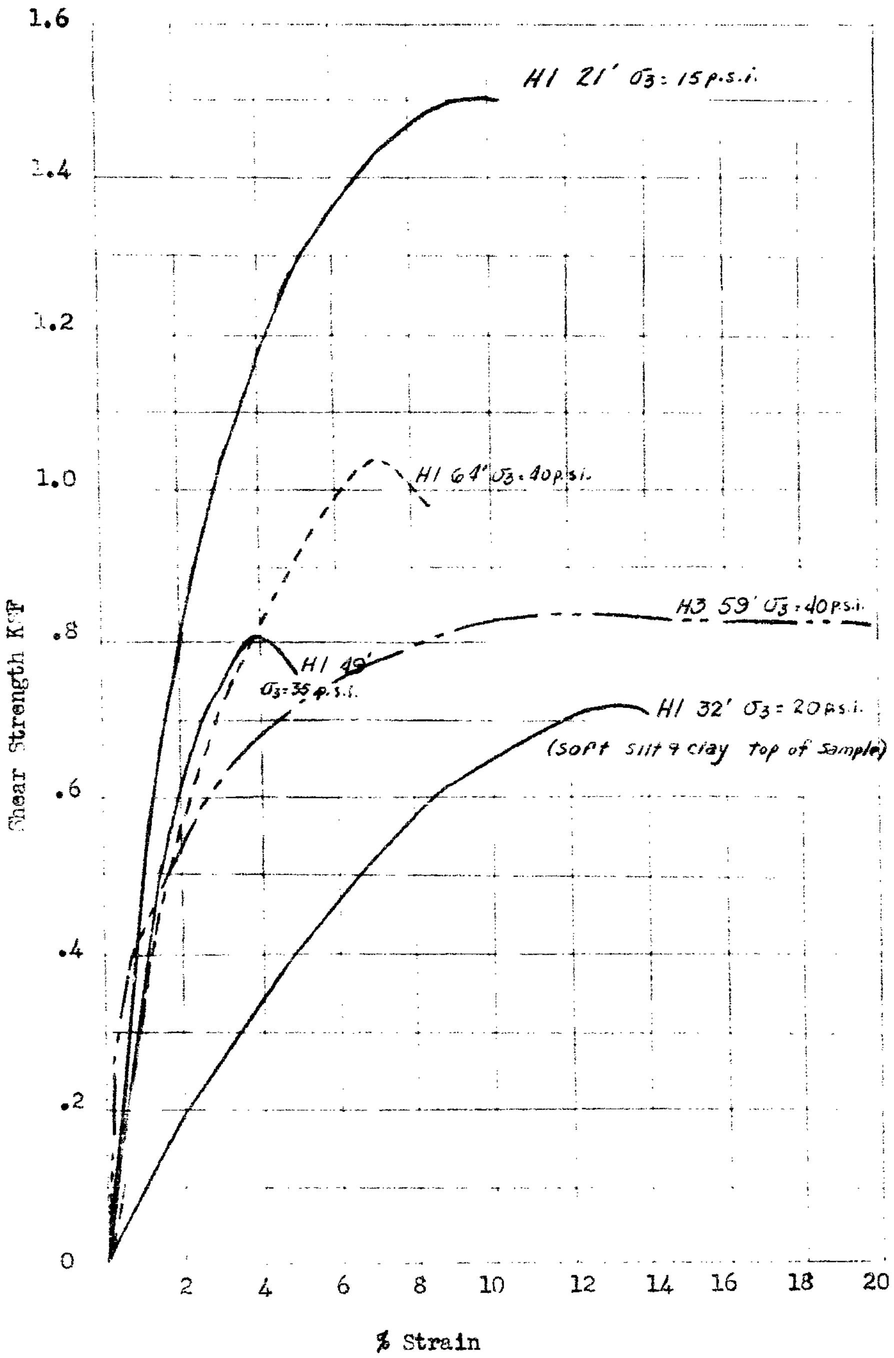
LEGEND
- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION (Qu)
- VANE TEST (C) AND SENSITIVITY (S)
- NATURAL MOISTURE AND LIQUIDITY INDEX
- LIQUID LIMIT
- PLASTIC LIMIT

PROJECT Proposed bridge site
LOCATION Gananoque River Hwy. #32
HOLE LOCATION See plan
HOLE ELEVATION AND DATUM Cone A & B - 268.8

BOREHOLE NO. Cone A & B
FIELD SUPERVISOR KP
DRILLER EA
PREP. KP

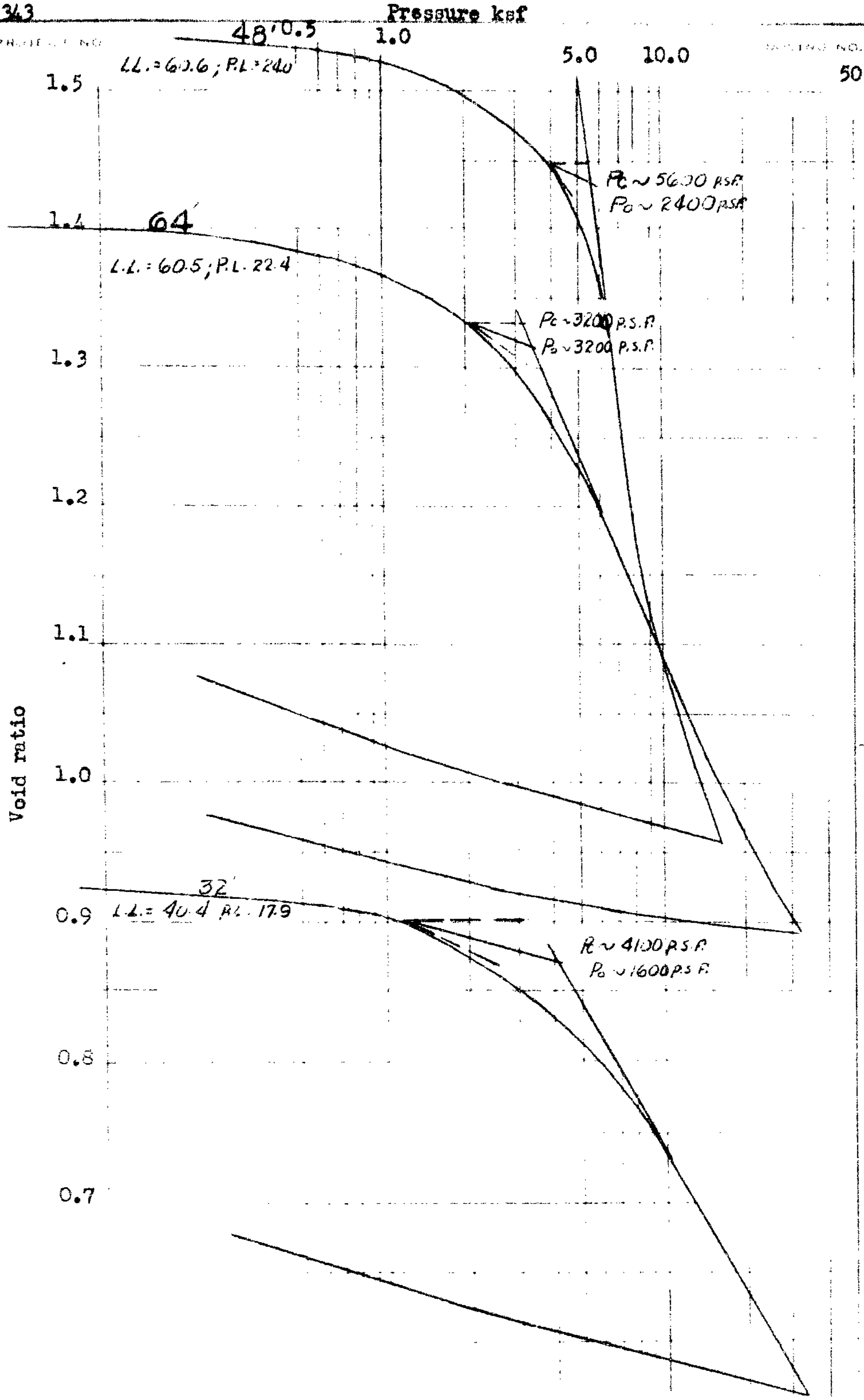
SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE P.S.F. / BLOWS FT.	CONSISTENCY / MOIST. CONTENT - % DRY WT	SAMPLE	NATURAL UNIT WT. P.C.F.
	River surface	268.8	0				
	Depth water Cone A - 1' Depth water Cone B - 12'						





STRESS STRAIN CURVES FOR UNDRAINED TRIAXIAL TESTS

J343 PROJECT NO. DRAWING NO. 9



CONSOLIDATION TEST RESULTS HOLE 1
GANANOQUE

WILLIAM A. TROW AND ASSOCIATES