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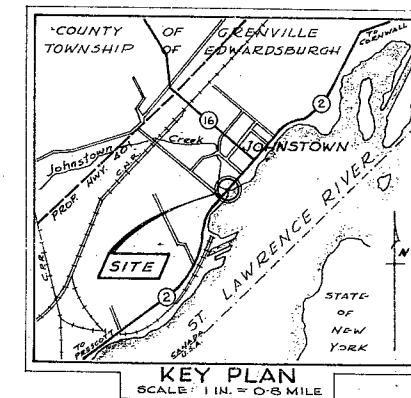
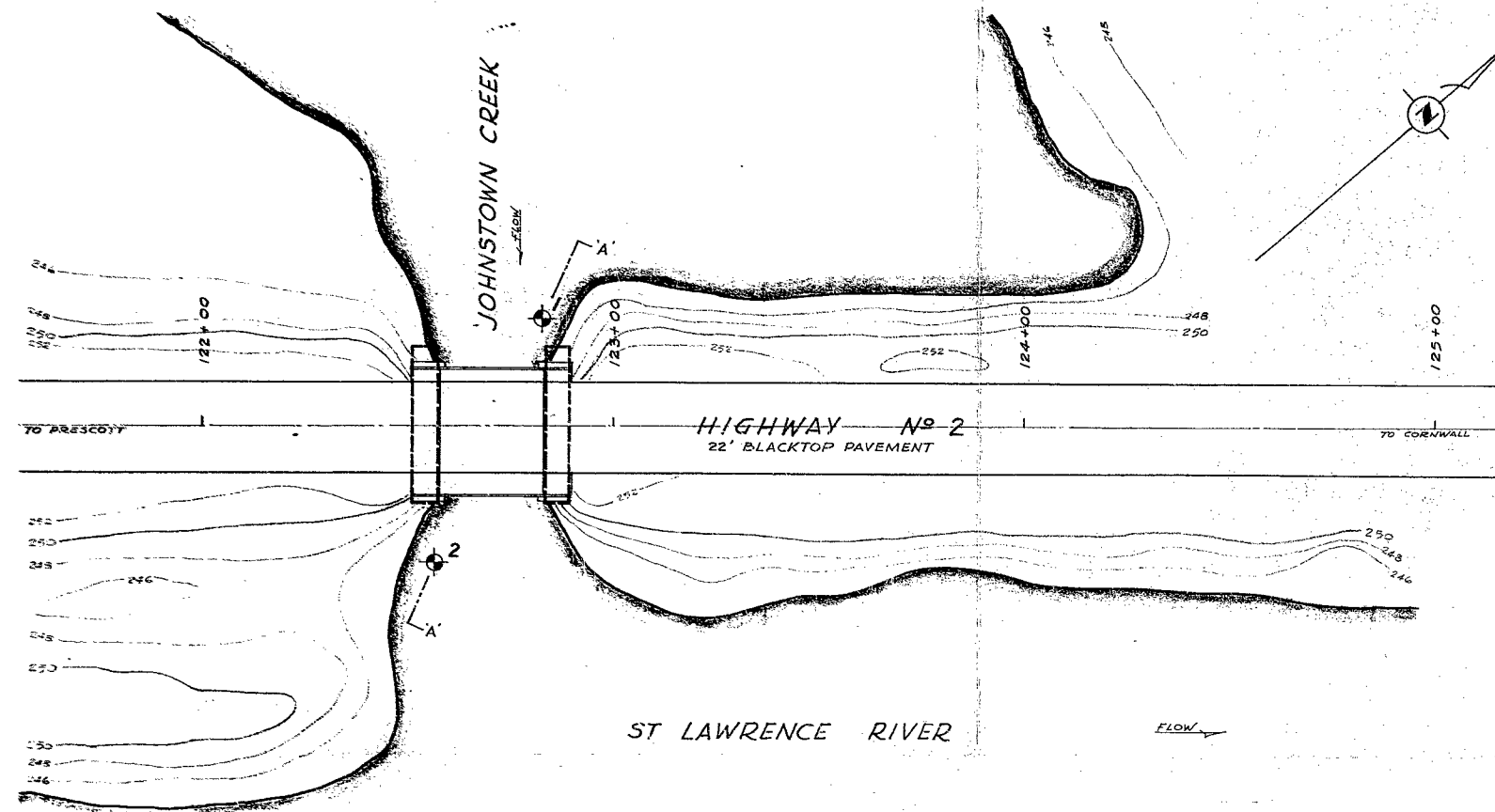
W.P.#60-62

HWY.#2 &

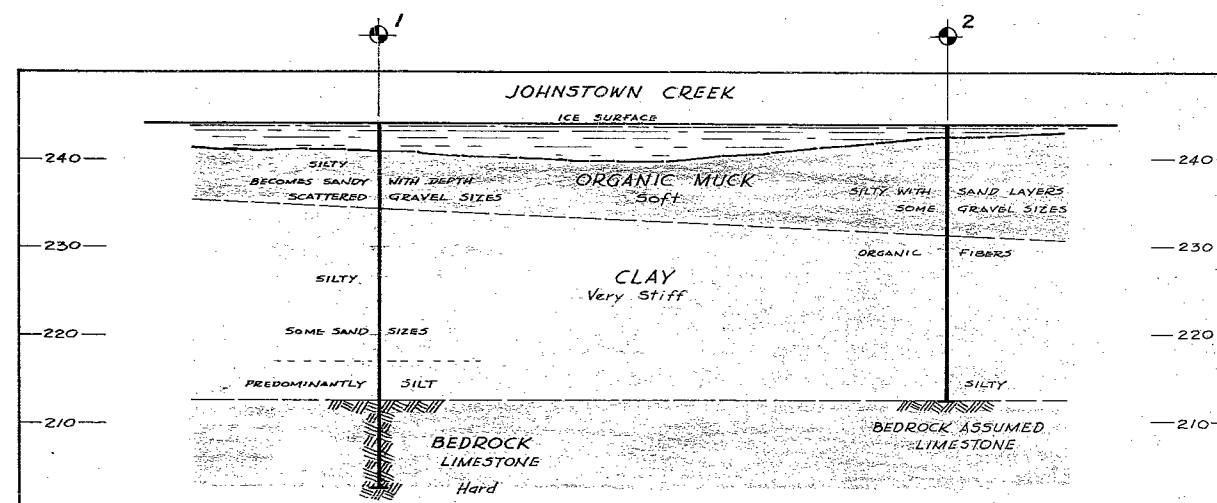
JOHNSTOWN

CREEK,

JOHNSTOWN



LEGEND			
BOREHOLE			
NO	ELEVATION	STATION	OFFSET
1	244.1	122+82.5	26' 2" LT.
2	243.9	122+56.5	33' RT.



W. A. TROW & ASSOCIATE, LTD.			
FOUNDATION INVESTIGATION			
PROPOSED BRIDGE REPLACEMENT			
JOHNSTOWN CREEK &			
KINGS HIGHWAY No. 2			
JOHNSTOWN - ONTARIO			
PROJECT NO. J 1015	W.P. NO. 60-62	DATE JAN. 63	DWG. 1

Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

Attention: Mr. S. McCombie.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

January 28, 1963.

FOUNDATION INVESTIGATION REPORT BY -
Wm. A. Trow and Associates, Limited,
Proposed Bridge Replacement, Johnstown
Creek, Hwy. No. 2, Johnstown, Ontario.
W.P. 60-62 -- Dist. #8.

Attached, we are forwarding to you, the above-mentioned report submitted by the Consultant, W. A. Trow & Associates of Toronto.

We have reviewed the report and have found the factual data well presented. In connection with the recommendations concerning the foundation, we would like to make the following comments:

A load of 60 - 70 tons can be attributed to individual steel piles. This figure is based on an allowable stress on the steel of 7,500 p.s.i. Our experience shows that an adequate factor of safety is incorporated if such loads are used.

For the calculation of the earth pressure acting on the back of the abutment, a value of 0.3 for the coefficient should be used unless a rigid frame structure is designed. In all other cases, a certain freedom of movement of the abutments is possible and, therefore, active earth pressure conditions would apply.

Soft muck can be ignored because the backfill along the entire abutment back face should be of granular material.

It is understood that the grade will be raised four feet. Although most of the settlement due to the present fill should have already taken place, there is no doubt that additional settlements can be expected due to the new fill. Consideration should, therefore, also be given to the alternative of removing - i.e., excavating the soft material for a distance away from the bridge and thus eliminate the unpleasant settlements and constant maintenance.

cont'd. /2 ...

Mr. A. M. Toye, Bridge Engr.,
Attention: Mr. S. McCombie.

January 28/63

It is suggested that some additional information on the subsoil stratification along the road's centre line be gathered before a final decision on this problem is reached. The Foundation Section is already in the process of doing this, and as soon as the information becomes available, will forward it to you for further consideration.

AGS/MdeF
Attach.

cc: Messrs. A. M. Toye (2) ✓
H. A. Tregaskes
H. D. McMillan
J. Ford
E. A. Cash
J. E. Gruspier
T. J. Kovich
J. Roy
E. R. Saint
F. Norman
A. Watt

Foundations Office
Gen. Files.

A. G. Stermac,
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

63 F 214 C

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

BA1578

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

1850 JANE ST.,
WESTON, ONT.
CH. 1-4644

Project: J1015

January 21, 1963

Mr. A. Rutka, P.Eng.,
Materials & Research Engineer,
Materials & Research Section,
Department of Highways of Ontario
Parliament Buildings, Toronto

Attention: Mr. A.G. Stermac

Re: Foundation Conditions - Proposed Bridge Replacement
Johnstown Creek Hwy. 2, WP 60-62

Dear Sirs:

We have completed our investigation of foundation conditions at this bridge replacement site which is located on Hwy. 2, near the west limit of Johnstown, Ontario. The field work involved in this project consisted of two cased borings made adjacent to diagonally opposite corners of the existing small bridge.

Our observations and recommendations arising out of this field study are outlined briefly in the following sections.

SITE

The ground in the vicinity of this crossing is flat and poorly drained. Johnstown Creek empties into the St. Lawrence river at this bridge location and the easterly approach to the existing structure takes the form of an earth causeway, which rises about 7 feet above the marshy water surface. The west approaches also have been raised to meet the bridge deck.

During spring flooding the water level reportedly rises to within 2 feet of the underside of the bridge.

Bedrock is said to be at a depth of 45 feet at the Texaco Station about 600 feet to the east. The piers of the International Bridge, some 1000 feet to the east, bear on rock at a depth of 22 feet, according to local residents.

SUBSOIL

The subsoil underlying this site is described in the two borehole logs, Dwgs. 2 and 3, and in the interpreted stratigraphical profile of Dwg. 1. It is seen that soft, silty, peaty mud with some sand layers extends below the shallow creek bed down to depths ranging from 9½ to 12½ feet from the water surface. This material had a measured shear strength in the order of 750 psf, according to field vane tests in hole 2, but this result probably is too high because of the layers of sand in the mud.

Below this material the natural soil consists of very stiff grey marine clay. According to laboratory and field vane tests, this material has an undrained strength in excess of 2200 psf. It continues to bedrock, which was established at El 212.4 feet, or 31 feet below the water surface. Bedrock consists of dense competent limestone of the Paleozoic Cambrian system.

FOUNDATION REQUIREMENTS

Two foundation alternatives appear to be applicable for this bridge replacement project. One involves the use of end-bearing piles, - either cylindrical or H piles, - driven to refusal on bedrock which lies about 31 feet below the water surface. The other alternative is to support the bridge on simple footings bearing near the surface of the very stiff marine clay ranging from 10 to 13 feet below the water level. In both operations the base of the footings will be well below the water surface, although the required excavation depth into the creek bed with the pile scheme should be much less than with the footing proposal.

Since the bedrock is quite hard and competent, end-bearing piles should encounter refusal immediately after it is contacted. The permissible loading on the piles should equal their safe structural capacity when considered as short columns. An ultimate bearing stress at the pile tips at least equal to 13,000 psi should be available. This is the approximate stress at failure recorded during H pile tests on the much softer Dundas Shale of the Toronto area.

The footing proposal will require excavations to be made through the upper 10 to 12 feet of organic mud and sand down to the clay. In order to accomplish this, steel sheet piling could be driven down about 5 feet into the clay, around the perimeter of the footings. Excavation to the clay surface can proceed inside this sheeted area. It will be necessary to brace the sheeting internally and also to install horizontal struts between the two cells in order to support the slight earth pressure unbalance resulting from this construction. This latter requirement can be eliminated if the existing approach fill is removed immediately adjacent to the abutments.

After the mud and water has been removed from the enclosed areas and the very stiff clay has been exposed, it should immediately be covered with a layer of concrete in order to provide a clean working surface. After this concrete has hardened, footing construction can proceed.

The safe net bearing value to apply to the clay is determined from the expression:

$$q = \frac{CN}{F}$$

where

C, the undrained shear strength, is conservatively taken to be in the order of 2250 psf according to laboratory and field tests

N is a bearing capacity factor which, for the footing shape, bearing depth and sheet pile confinement should be at least equal to 6

F=3 is the recommended factor of safety

Solving this expression, the safe net bearing value is determined to be 4500 psf. The settlement resulting from this stress application will be well within tolerable limits for this bridge.

The magnitude of earth pressure exerted against the walls of the abutments will be determined, to some degree, by the bridge scheme used. In both proposals, granular fill will be used as backfill, at least down to water level, and, since the structures will be braced by the bridge deck, movement can occur only by rotation about the base. However, with batter piles incorporated in the foundation scheme, - or if the footings are set in the very stiff clay, - there should be ample resistance to movement at the footing level, and therefore an "at rest" rather than active earth pressure condition should develop. For compacted granular fill, the "at rest" earth pressure coefficient will be approximately equal to 0.4. Near the sides of the bridge the pressure will be less than this, because of the three dimensional spread of load.

It is recommended, therefore, that the abutments be designed to resist the "earth pressure at rest" force exerted by the existing fill and by the additional fill, associated with the revised road grade. This assumption is believed to be conservative. In the footing scheme, an earth pressure at rest coefficient equal to unity should be assumed for the organic mud lying above the stiff clay. It is estimated that some of this compressible mud must still remain under the existing bridge approaches, since the roadway is somewhat settled, even now, below the level of the bridge deck. Motorists and truckers complain of a bump at the transition to the bridge.

For purposes of design, it could be assumed that approximately 5 feet of mud remains above the clay under the approaches, and that the horizontal earth pressure coefficient, within this depth of mud, is equal to unity. With sheet piling driven along the front face of the footing 5 feet into the very stiff clay, it can be shown that there will be ample resistance and moment generated along this face and under the footing to resist this conservatively assumed pressure distribution. If positive measures are taken to excavate the mud down to the clay behind each abutment and to replace it with granular material, an even lower earth pressure will be exerted.

For the pile foundation scheme, it is assumed that batter piles will be incorporated in the design in order to provide for traffic impact forces. It is assumed also that the base of the pile caps will terminate in the mud or fill well above the clay. Consequently, there will be less earth pressure exerted against the abutments and it will essentially take the granular distribution referred to above. The batter piles will resist this horizontal thrust from the fill.

If mud underlies the fill approaches, it could be argued that the weight of the 4 feet of additional fill, associated with the raising of grade, will produce an earth pressure in this mud around the abutment piles. This horizontal pressure should become critical when the surcharge weight approaches a value of 4 times the cohesion of the mud, as indicated in the expression:

$$Y_h = 4c^*$$

The magnitude of fill pressure, Y_h , at the top surface of the mud will be approximately equal to 1600 psf after the new fill has been added. The undrained shear strength, c , needed to balance this force, from the above expression, is 400 psf. According to field vane tests, the strength of the mud is in excess of this critical value. This should be the case, particularly, under the existing approach fill, since any remaining mud must be completely adjusted to present loadings. Therefore a higher strength should be available in this mud, - to resist the thrust from the additional 4 feet of fill, - than is inferred from the vane test results.

It is concluded, therefore, that there will be no unbalanced earth pressure against the piles because the surrounding soil will have ample strength to absorb it. Consequently, the batter piles will be required only to resist the pressure exerted against the abutments. The conservative assumption for this pressure has been outlined above.

Although this bridge structure is not large, it is felt that some examination and estimation of the magnitude of horizontal earth pressure was warranted, since the problem cannot be overlooked in this marshy ground. The method of approach is approximate and it is believed that the pressure assumptions are quite conservative. However, in either bridge foundation scheme there should be adequate resistance to accommodate even these assumed pressures.

* "Soil Mechanics in Engineering Practice", Terzaghi & Peck, Pgs. 185-6

Some small settlements of the existing roadway approaches over the marshy ground should be anticipated after the grade has been raised if organic mud underlies the existing fill. Consequently, some long term maintenance of the surface may be required.

The foregoing comments endeavour to anticipate the foundation requirements for the replacement of this creek crossing. If you have any queries on this subject, or if other thoughts come to mind after you have reviewed this information, we shall be pleased to discuss them with you.



WAT/gc
Encls.

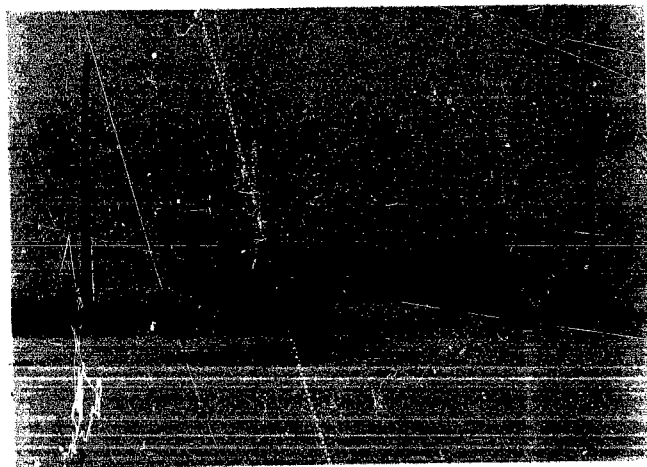
Yours very truly,

W. Trow

William A. Trow, P.Eng.



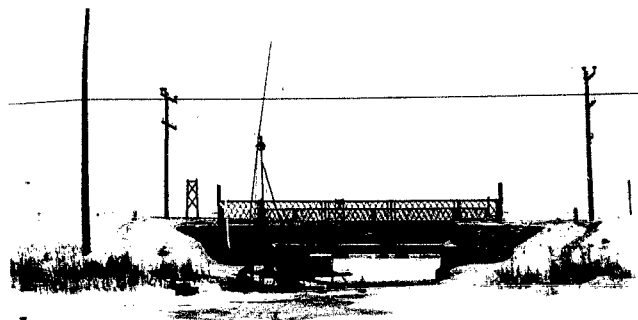
Looking East, Drill on B.H. 1



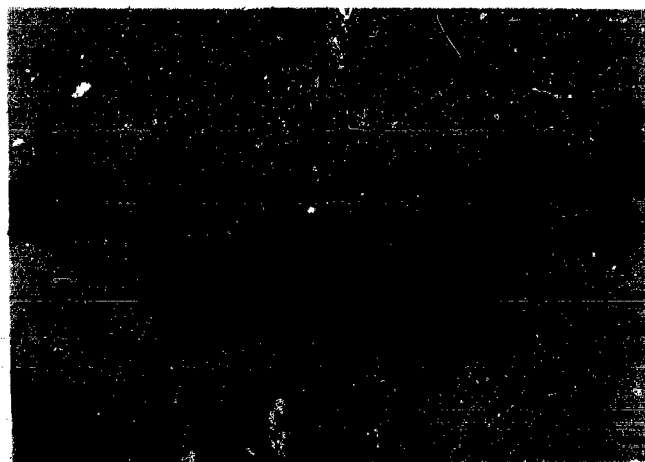
North Side of Existing Bridge, Drill on B.H. 1



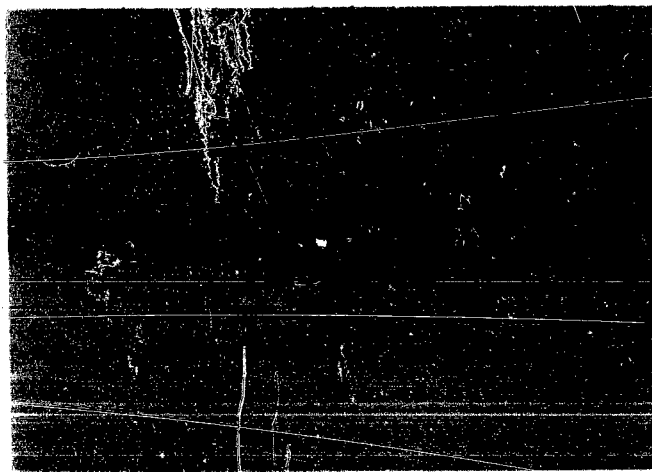
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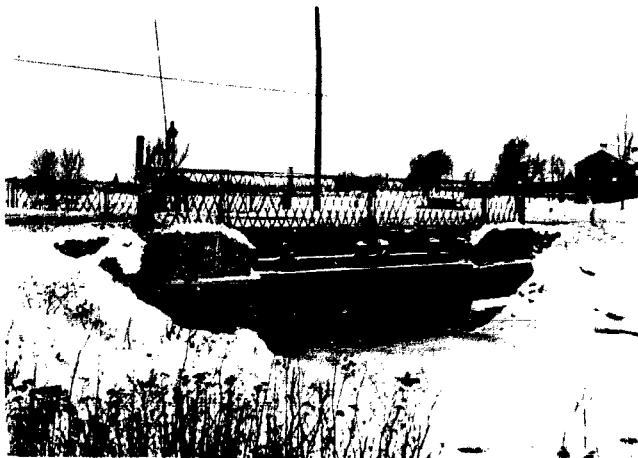
North Side of Existing Bridge, Drill on B.H.



South Side of Existing Bridge



Looking West, Drill on B.H. 2



South Side of Existing Bridge



Looking West, Drill on E.H. 2

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING NO. 2
PROJECT NO. J1015

LEGEND

PENETRATION RESISTANCE

- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 2" DIA. CONE

SHEAR STRENGTH

- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE
- UNCONFINED COMPRESSION
- VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

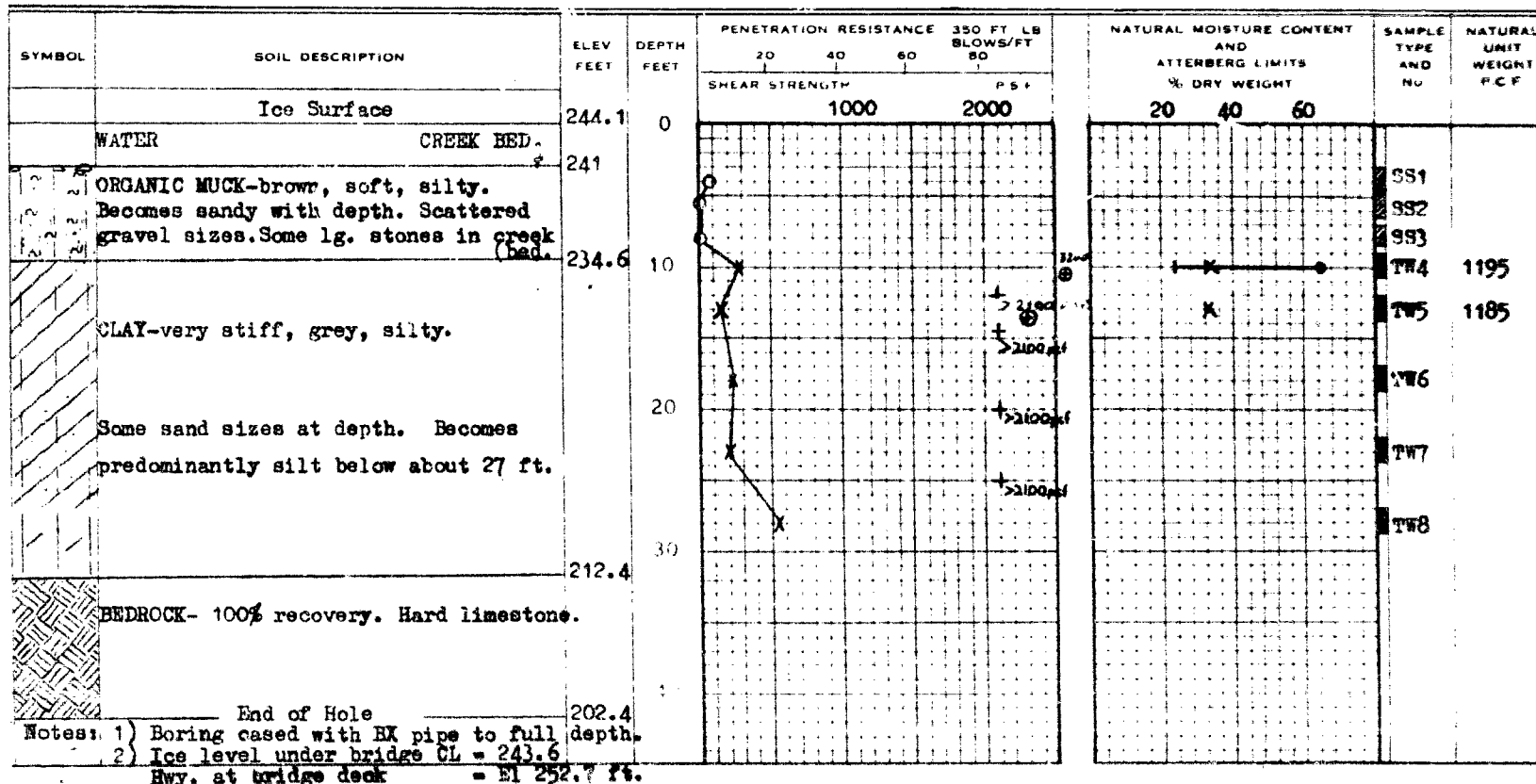
ATTERBERG LIMITS

- LIQUID LIMIT
- PLASTIC LIMIT

SAMPLE TYPE

- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

BOREHOLE NO. 1
PROJECT Bridge Site, Johnstown Creek & Hwy. No. 2
LOCATION Johnstown, Ontario
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 244.1 ft.
DATUM See Dwg. 1.



SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING NO. 3
PROJECT NO. J1015

LEGEND

BOREHOLE NO. 2
PROJECT Bridge Site Johnstown Creek & Hwy. No. 2.
LOCATION Johnstown, Ontario
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 243.9 ft.
DATUM See Dwg. 1.

2 OD SPLIT TUBE
2 ID SHELBY TUBE
2 DIA CONE

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) ⊕

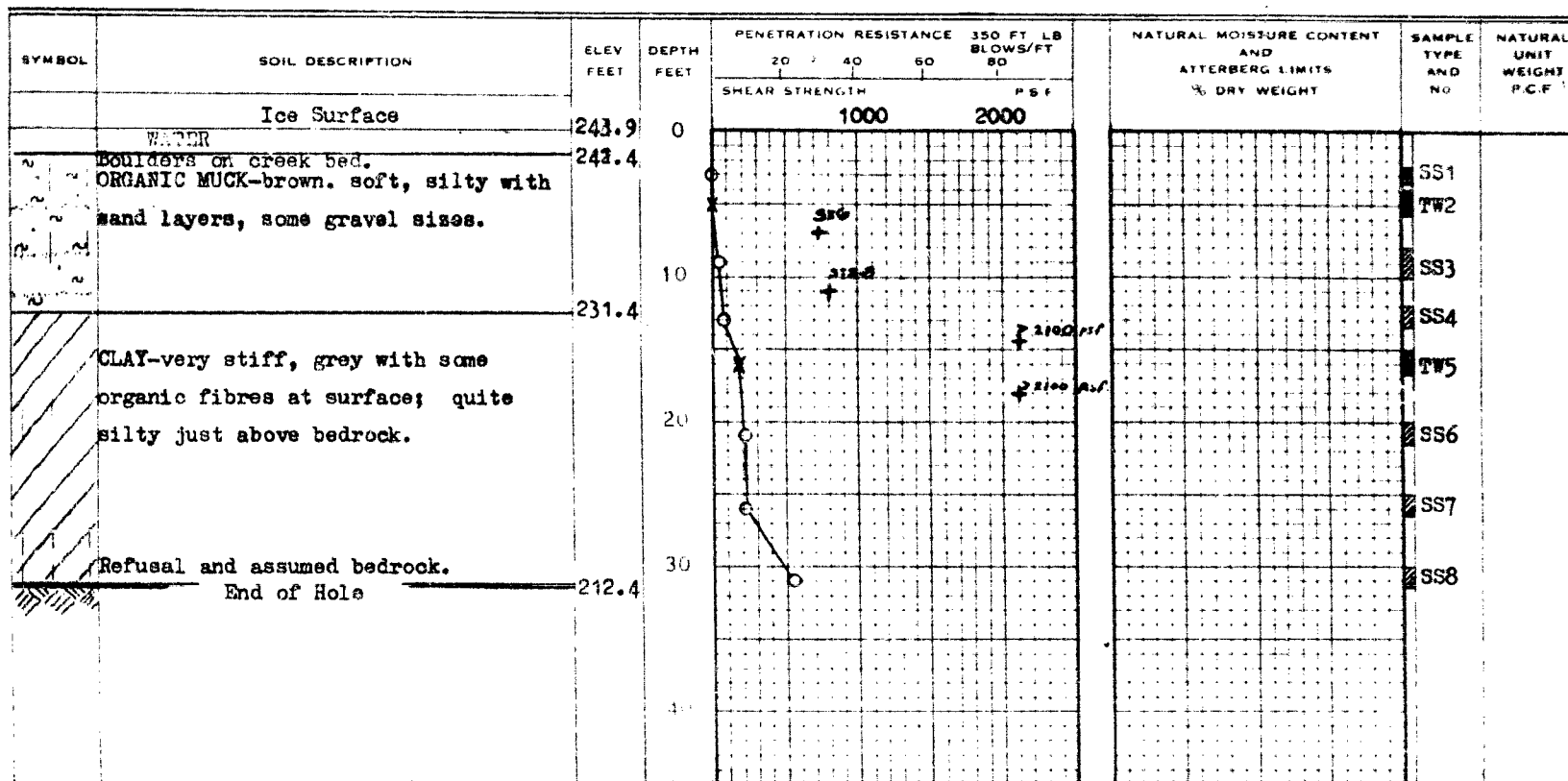
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

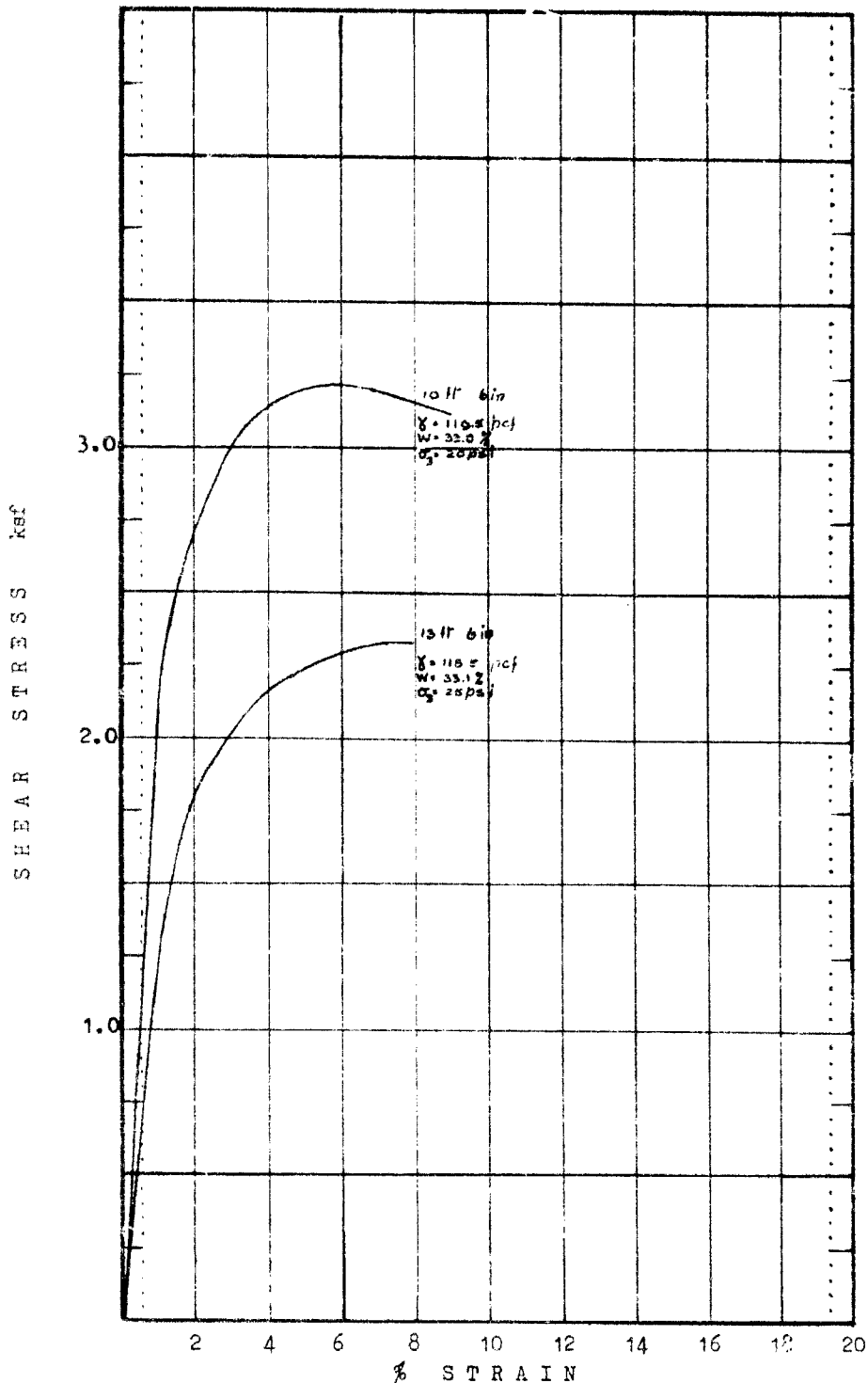
ATTERBERG LIMITS

LIQUID LIMIT
PLASTIC LIMIT

SAMPLE TYPE

2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE





UNDRAINED TRIAXIAL TEST RESULTS