

BA 1616

WILLIAM A. TROW AND ASSOCIATES LTD.

SILT INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

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

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

Project: J1022

February 8, 1963

STRUCTURE SITE No. 2A-246

McDowell and Jewitt,
Engineering and Surveying,
92 Kent Street
Simcoe, Ontario

Attention: Mr. W.C. McDowell, P.Eng.

Re:

Foundation Conditions
Proposed Bridge Reconstruction over Spittler Creek
Lot 20, Concessions VII & VIII, Twp. S. Norwich, Co. Oxford

Dear Sirs:

This report describes the foundation conditions existing at the site of the proposed bridge replacement. The information contained in this submission essentially confirms advice given to you immediately following completion of the on-site work.

The significant observations and recommendations arising out of this study are summarized briefly as follows:

1. The proposed bridge replacement is located at the crossing of the Spittler Creek valley by a gravel concession road. The valley at this point is approximately 800 feet wide, and the level of the flood plain lies about 40 feet below the ground elevation of the surrounding rolling country.

It is understood that improvement of the channel alignment may result in the new bridge being slightly east of the old bridge, but without any change in the road alignment.

2. The soil underlying the thin flood plain alluvium consists of stiff lacustrine clay and silty clay in which thin seams and beds of silt, silty clay and clay alternate rapidly. A transition to a series of dense silts and fine sandy silts occurs at a depth of about 29 to 33 feet, by an increase in the number and frequency of the silt seams. The clays were observed to be stiffer and more competent towards the top of the stratum.

3. It is recommended that the bridge be supported on spread footings placed just within the upper limit of the clay series. At this elevation advantage is being taken of:

- a) the higher bearing strength of the uppermost clay
 - b) the reduction in required total excavation, and in excavation below water level.
- and
- c) the minimizing of potential disruption or disturbance of the footing beds during excavation, by uplift within the coarser silt seams.

A founding elevation at El 61.0 feet is recommended, with a safe net bearing value of 4000 p.s.f. applied. At this founding level scour protection in the form of sheet piling would be required, although scour is not considered to be a serious problem in this stiff clay.

If footings were lowered to El 56.0, where no scour protection would be required, the safe bearing value must be reduced to 2400 p.s.f. approximately.

4. Although scour within this clay soil should be limited in extent, all foundations placed above El 56.5 should be protected against possible erosion and undermining. Such protection would be most readily provided by driving sheet piling around the face and ends of each abutment footing to a depth of 4 feet below present maximum river bed level, or to about El 56.5. The upper ends of the piling should be anchored to the footing. This sheet piling could be continued along the toes of the approach fills, if necessary to prevent undermining of the fill by lateral stream shift. Alternatively, protection of the approach fills could be provided by placing rip rap over a sand and gravel blanket, along the toe and side slopes.

5. Alternative foundation consideration can be given to friction piles of the Class B timber type. A working capacity of 20 tons per pile should be reached when the piles are driven to approximate El 40, or 20 feet below the river bottom. This resistance will be developed by a combination of end-bearing pressure in the dense silt and sand below El 41 feet, and by adhesion of the clay on the shaft itself.

6. It is reported that, although the existing bridge has not been overtopped during peak spring run-off the approach fills have been flooded to depths of 3 or 4 feet. This considerable depth

probably represents backwatering from the culvert restriction at the railway embankment some 1000 feet downstream. Additional fill will therefore be required to prevent any recurrence of this roadway inundation.

Some limited settlements may be expected to occur under this additional loading, but the magnitude of such settlement is estimated to be very small. No differential settlement between the bridge structure founded on spread footings, placed high up in the clay, and the adjacent abutment fill is to be expected.

The factual information and data forming the basis for these comments is outlined in more detail below.

SITE

The bridge site is located on Spittler Creek, a left bank tributary of Big Otter Creek, at a point some 3 miles upstream from the confluence of the two streams. An existing concrete abutment - steel girder-timber decked bridge carries the gravel concession road across the creek. The roadway embankment approaches to the bridge across the flat valley floor are approximately 350 feet long on the west side and 450 feet long on the east side. Severe flooding of these embankments up to depths of 4 feet has been reported in the past, due possibly to backwatering above the railway embankment some 1000 feet downstream.

The flood plain of the creek occupies the whole valley floor and is some 800 feet wide. The valley sides rise steeply from this flood plain up to the level of the surrounding rolling plain. The total rise is in the order of about 35 to 40 feet.

The west side of the valley opens out sharply immediately downstream of the concession road, where a small tributary valley enters.

It is understood that the proposed reconstruction of the subject bridge is to include improvement of the stream channel alignment.

FIELD AND LABORATORY WORK

A total of two sampled borings and one dynamic cone penetration test were completed during the field work. The equipment used was a trailer-mounted continuous flight machine auger rig. The holes were uncased.

Sampling was carried out in the two principle borings at regular intervals of 5 feet or less using a 2 inch O.D. split spoon sampler. Additional undisturbed samples were then recovered from the auxilliary holes by means of 2 inch I.D. Shelby tubes, at the elevation of the probable founding level. Two field vane tests were also carried out in hole 1.

The deep sample borings were terminated after only a limited penetration of the lower dense silt stratum because of a) the commencement of some cave-in of the holes at this depth and b) the consideration that simple footings at shallow depth or short friction piles would suffice to support the planned small bridge structure.

All sampling and field test data is presented in detail in the individual boring logs, Dwgs. 2 and 3. The borehole locations, and interpreted subsoil profile, are shown in Dwg. 1. Ground elevations were referred to the deck level of the existing bridge structure at centre/centre, which has been given as Elevation = 72.92 feet.

In view of the laminated and variable character of the upper subsoils a limited number of laboratory tests were carried out in order to appraise the competence of the clay stratum. The results of these tests are given in both the field logs, Dwgs. 2 and 3, and in the auxilliary test result sheets, Dwgs. 4, 5 and 6.

SOILS AND GEOLOGY

The subsoils underlying this bridge site and its surrounding area consist of a series of stratified clays and silty clays overlying a further stratified series of silts and sandy silts. Both soil types were probably formed as deltaic or lacustrine deposits during an early post glacial phase of the geologically recent Pleistocene Glacial Epoch. The transition from the predominantly clay series to the predominantly silt series occurs at depths of 20 to 25 feet below the valley floor.

Recent detrital alluvial deposits form a thin veneer across the floor of the valley, and make up the low flood plain terrace. These alluvial soils are primarily silty sands and gravels.

The limestone bedrock underlying this region was not encountered during the subject field work.

FOUNDATIONS

The principle foundation considerations and recommendations have been outlined in the opening paragraphs of this report, and therefore no purpose is served by a repetition of this information. However, some comments on the bases of our observations may be useful.

The recommended bearing pressures and founding elevations were determined from field penetration and cone resistance data, from critical visual examination of all the soil samples, and from laboratory tests on a limited number of selected samples. The safe net bearing pressure at El 61 feet is determined from the expression:

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

where: C=2000 psf is the undrained shear strength of the clay

N=6 approx. is the appropriate bearing capacity factor for the footing size and depth applicable here

F=3 is the recommended factor of safety

At greater levels the shear strength of the clay decreases somewhat although the decrease is not believed to be as great as is indicated by the test at 17 feet in hole 2. This sample was partially disturbed and possibly affected by numerous silt seams.

It appears from this examination that the most favourable founding conditions occur near the very top of the stratified clay series because of the following:

- a) The bearing capacity of the clay decreases somewhat with increasing depth.
- b) The number and individual thicknesses of silt seams within the clay also increases with depth. The likelihood of disturbance of the footing beds, occurring through ground water uplift and seepage within the silt seams, is therefore more likely to increase as the foundation depth and hydraulic head also increase.
- c) Quantities of excavation, both as total and below water level, are minimized.

The use of friction - and bearing piles as a foundation medium is suggested as a secondary alternative consideration. A total bearing capacity in excess of 20 tons for each pile should be developed in a combination of frictional and end bearing resistances, by driving Class B timber piles down to the dense silt and sand below Bl 42 feet.

Frictional resistance on the shaft has been calculated on the basis of the expression:

$$QF = \frac{I}{F} (P.L.C_A)$$

where:

QF = frictional load capacity

L = length of pile in contact with the clay = 20 feet approx.

P is the perimeter of a pile = 3 feet approx.

C_A is the adhesion of the clay on the shaft estimated to be 0.5 tsf

$F = 2$ is the recommended factor of safety

Solving, QF is determined to be 15 tons.

End bearing resistance in the sand is given approximately by the expression:

$$Q_E = \frac{A}{F} (0.3YDN_Y + YLN_Q) *$$

where:

Q_E = end bearing load capacity

Y = density of sand = 60 psf approximately

D = diameter of pile in feet = 0.66 for Class B timber piles

L = length of pile in the ground below stream bed = 20 feet approximately.

A = area pile tip = 0.35 sq. ft.

and $N_Y = N_Q$ are bearing capacity factors estimated to be = 200 for this situation

Solving, Q_E is determined to be 21 tons.

Therefore, total pile capacity, $Q_F + Q_E$ is approximately 41 tons.

* "Soil Mechanics in Engineering Practice" - Terzaghi & Peck, Pg.172

As stated in the summary, settlements under the bridge and fill loadings are not expected to be great, and should be less than 1 inch. The horizontal pressure exerted by the approximately 11 feet of approach fill against each abutment should be in the order of 1800 plf, assuming granular backfill and an active earth pressure coefficient of 0.25. Sufficient shearing resistance will be developed under the base of the footings to resist this force.

We shall be pleased to discuss any queries you may have on the information presented in this report. We sincerely regret the delay in the submission of this report to you.

Yours very truly,



William A. Trow, P.Eng.

JIM/gc
Encls.

BOREHOLE NO. 1
PROJECT Spittler Creek Bridge Reconstruction
LOCATION Two. S. Norwich, Co. Oxford, Ontario
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 72.2 ft.
DATUM See Dwg. 1.

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—

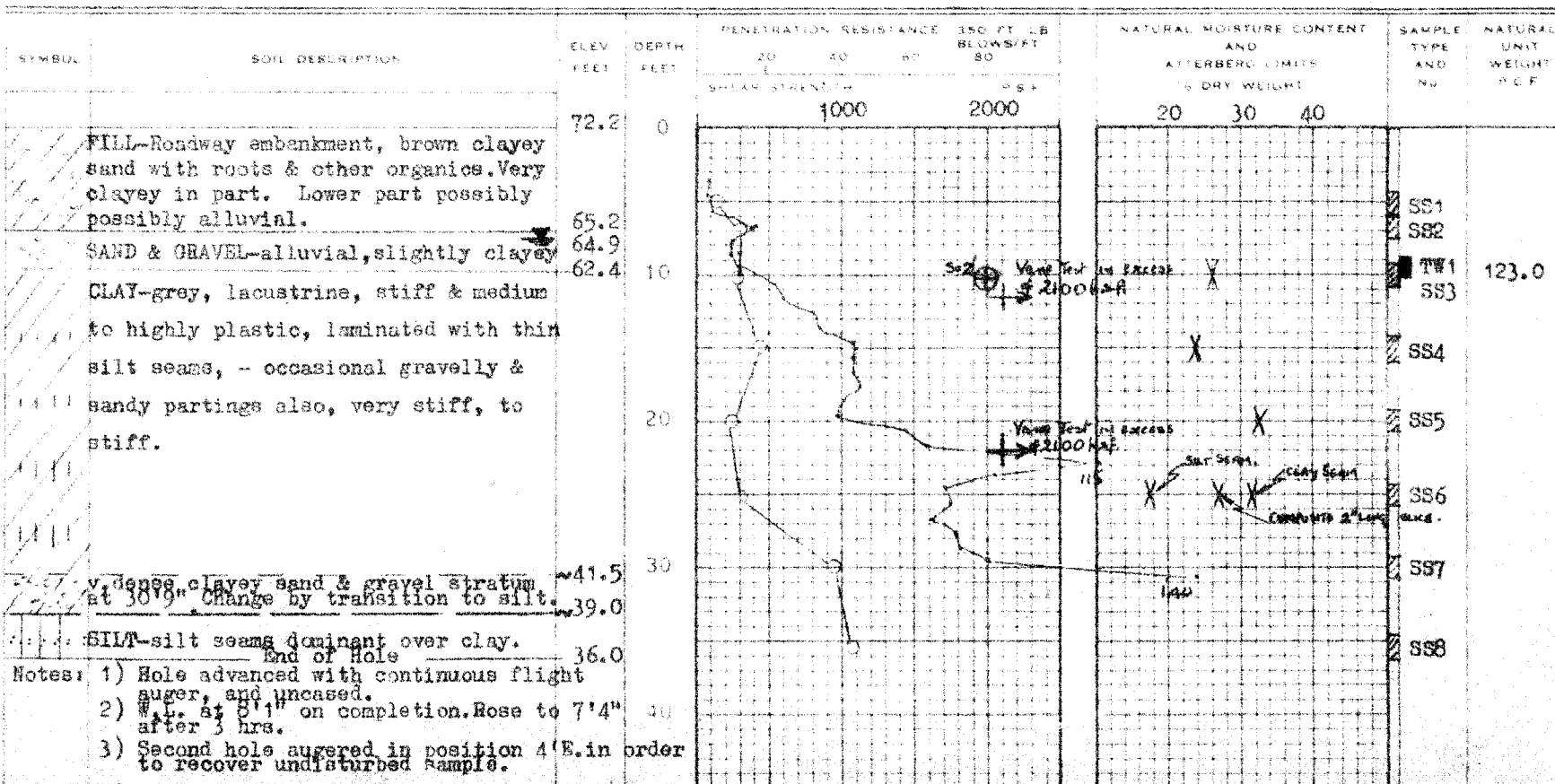
NATURAL MOISTURE CONTENT
AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT —○—
PLASTIC LIMIT ———

SAMPLE TYPE

2" O.D. SPLIT TUBE —○—
2" I.D. SHELBY TUBE —+—
3" O.D. SHELBY TUBE —⊕—



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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

LEGEND

DRAWING NO. 3
PROJECT NO. J1022

BOREHOLE NO. 2
PROJECT Spittler Creek Bridge Reconstruction
LOCATION Twp. S. Norwich, Co. Oxford, Ontario
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 72.8 ft.
DATUM See Dwg. 1.

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) +

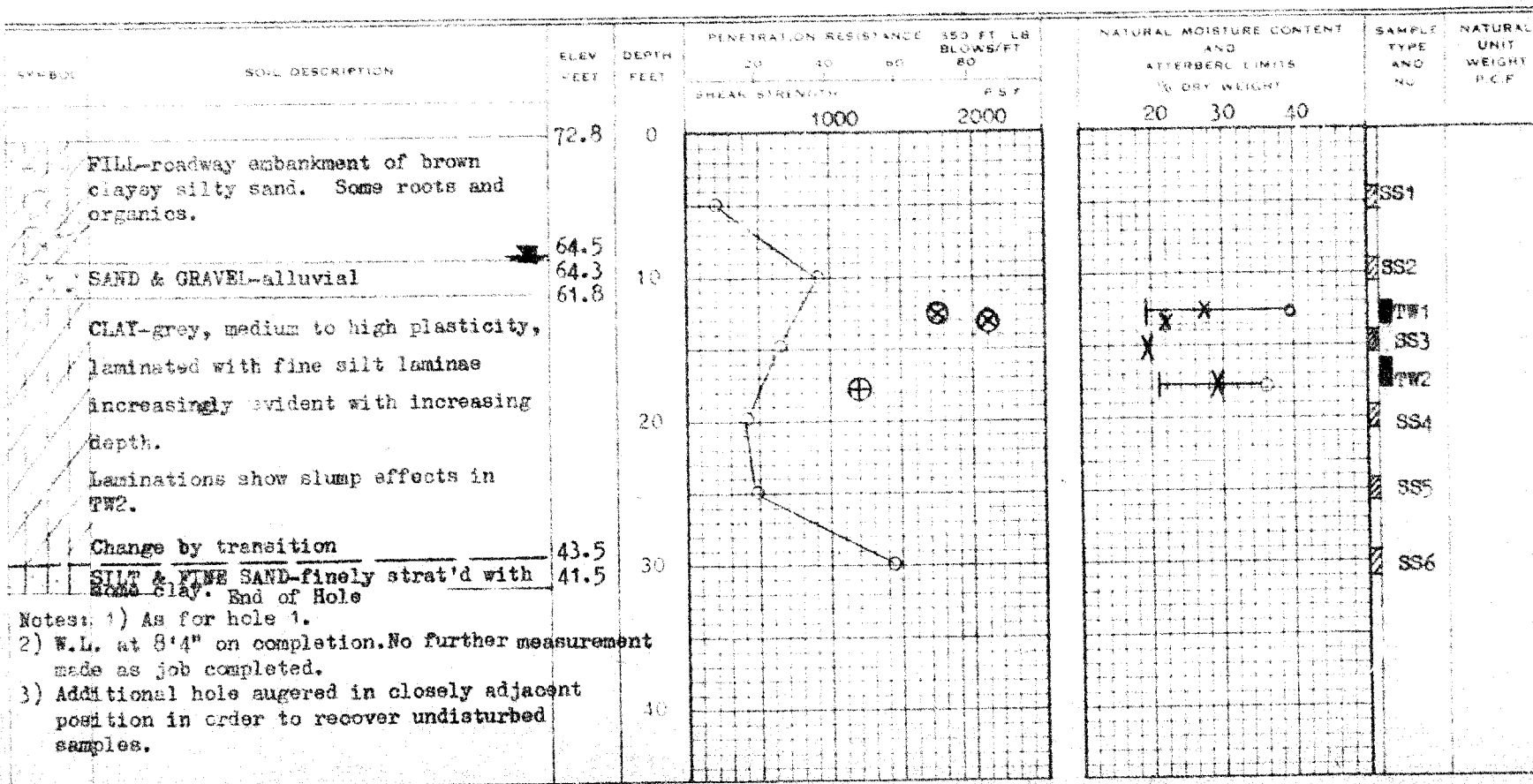
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS

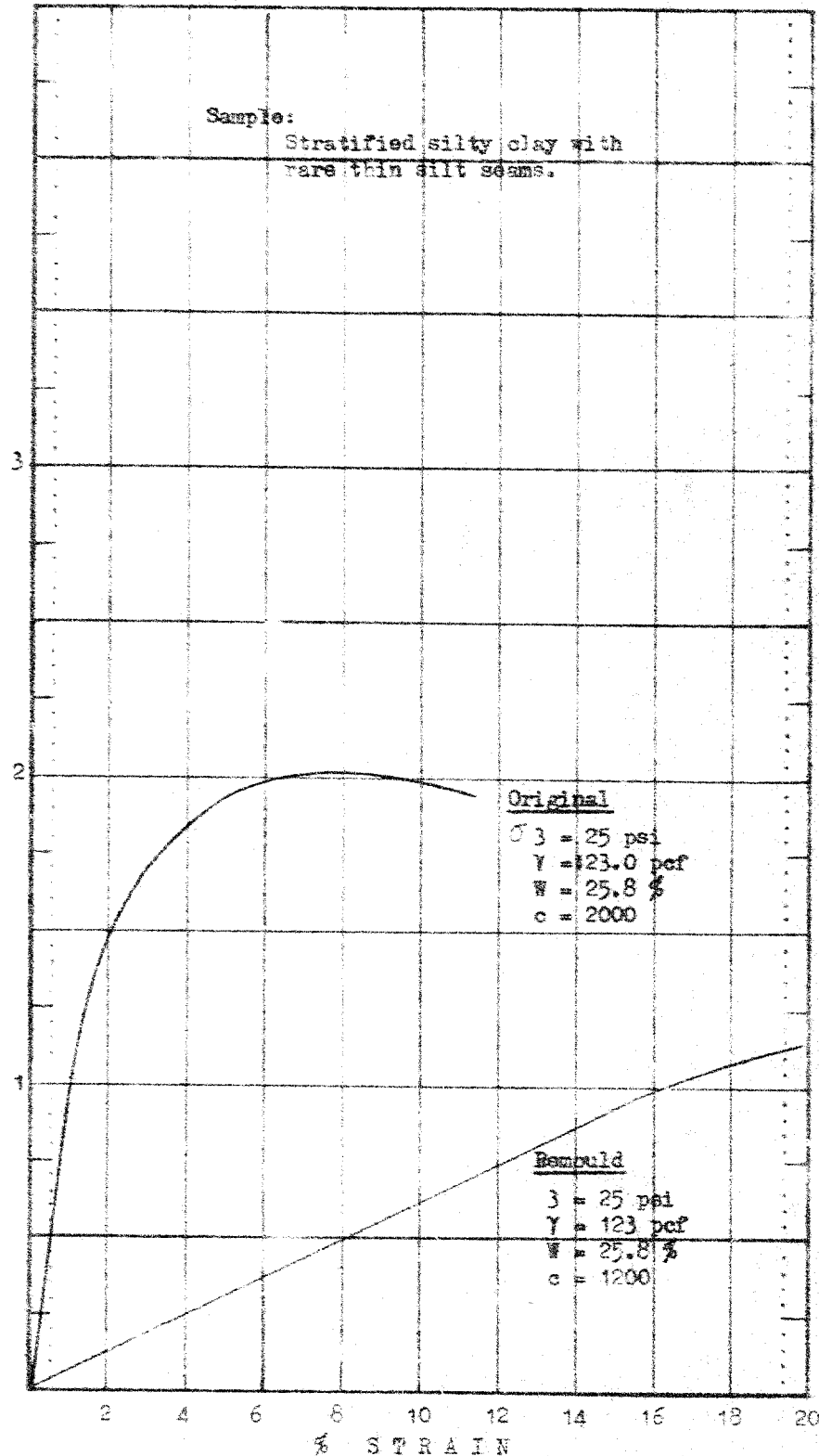
LIQUID LIMIT —○—
PLASTIC LIMIT ———

SAMPLE TYPE

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



S H E A R S T R E S S k s f

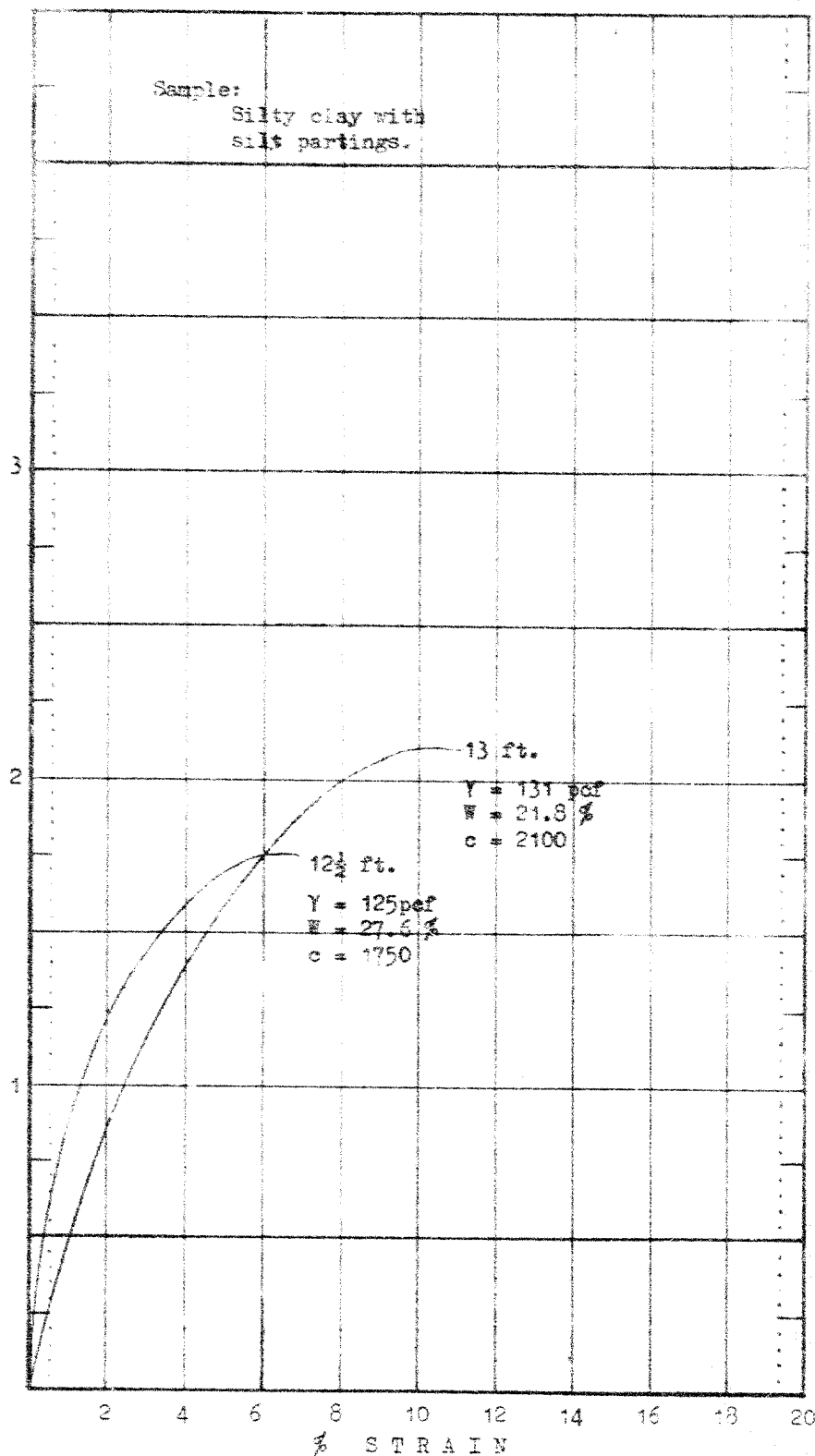


UNDRAINED TRIAXIAL TEST - HOLE 1A, 9 - 11 Ft.

UNDISTURBED REMOULDED SAMPLES

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S H E A R S T R E S S ksf



UNCONFINED COMPRESSION TEST RESULTS, HOLE 2 - 12 Ft.

SHEAR STRESS ksf

1.5

1.0

0.5

Sample:

Stratified silty clay
with rare silt seams
and very rare fine pebbles.

$\sigma_3 = 30$ psi
 $\gamma = 124.0$ pcf
 $w = 29\%$
 $C = 120$ psi

2

4

6

8

10

12

14

16

18

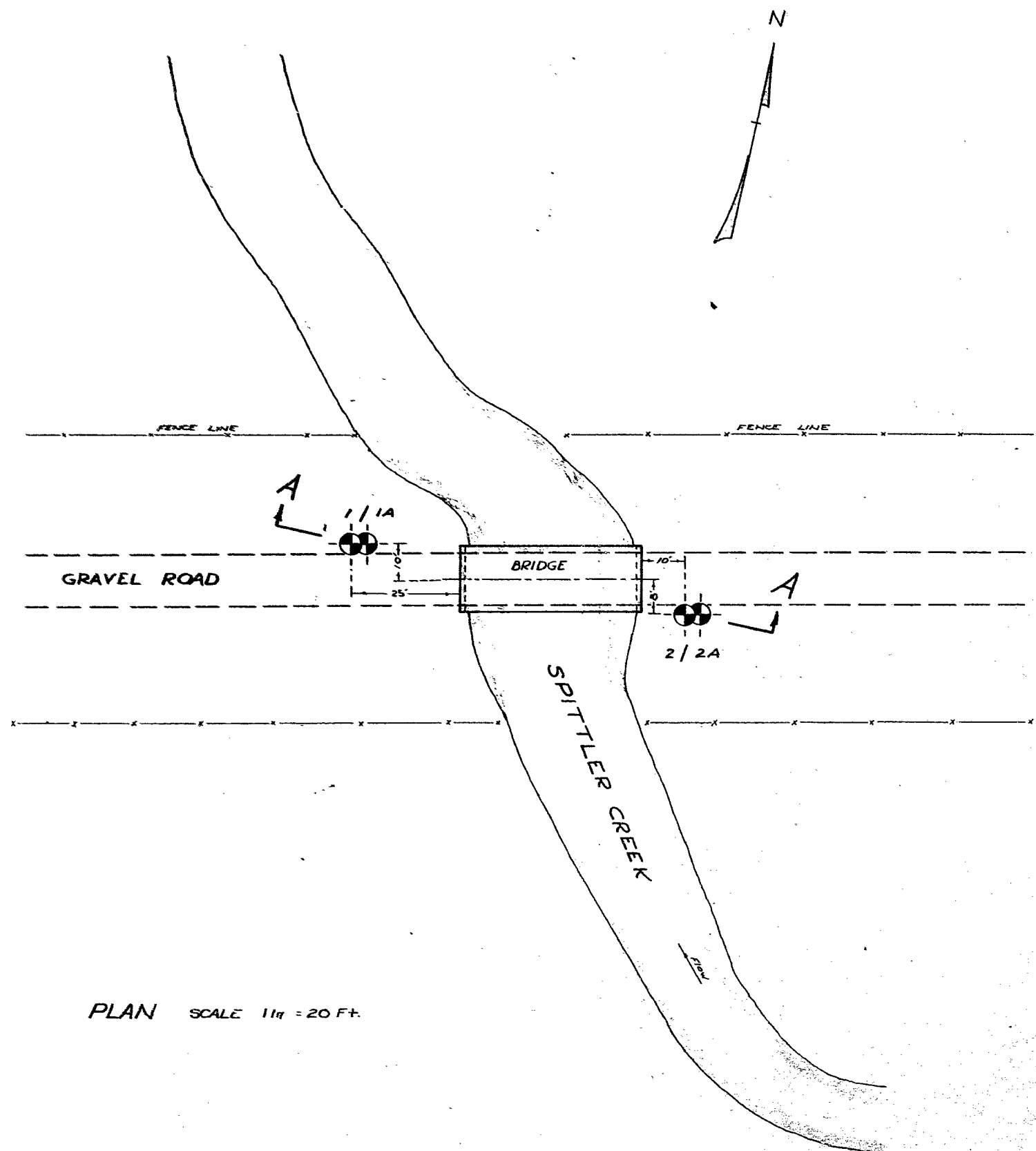
20

% STRAIN

UNDRAINED TRIAXIAL TEST

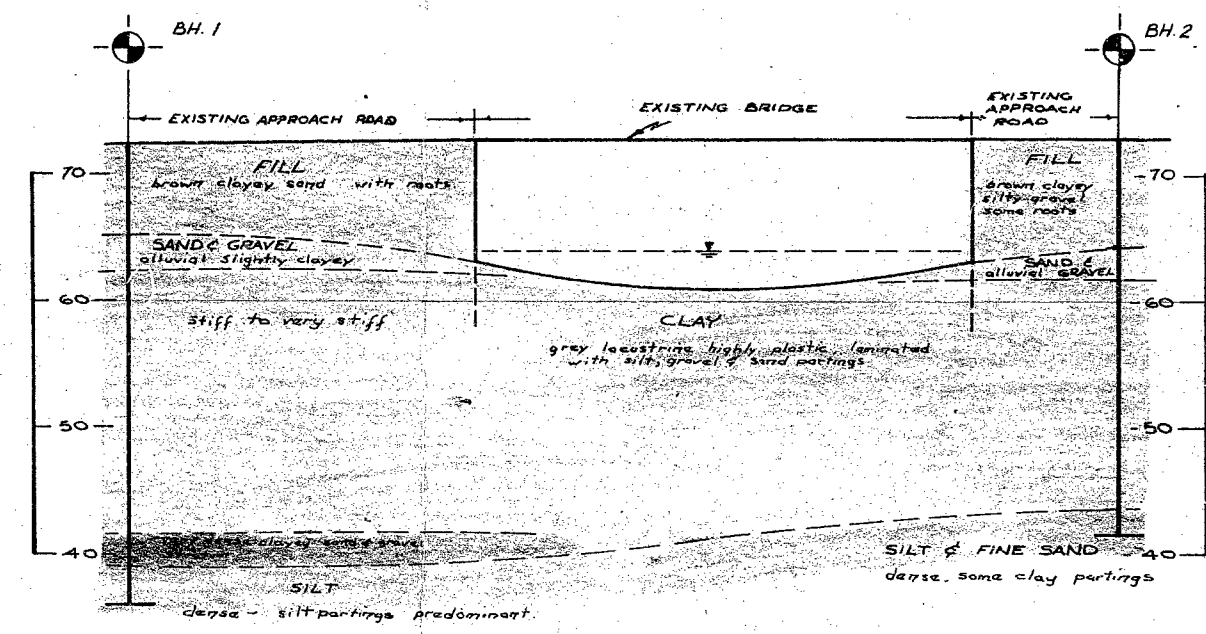
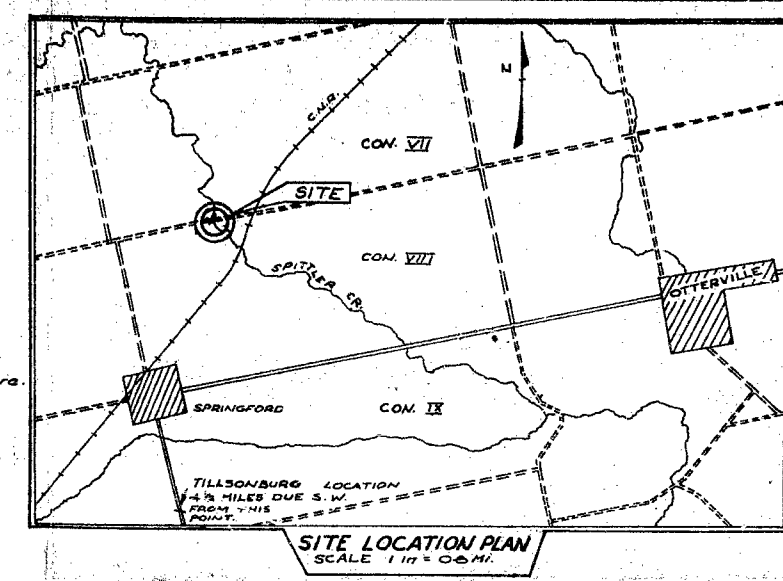
HOLE 2A - DEPTH 17 Ft. (Poor Sample)

#63-F-279M
BRIDGE OVER
SPITTLER CREEK
LOT 20. CON. VII & VIII
SOUTH NORWICH
TWP.



LEGEND
 BOREHOLE

NOTES
 Ground elevations referred to bridge deck level, centre/centre.
 Elevation provided by client as 72.92 Ft.
 Plan drawn from measurements taken on site.



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FOUNDATION INVESTIGATION		
PROPOSED BRIDGE REPLACEMENT CONCESSION ROAD OVER SPITTLER CREEK LOT 20, CON'S VII, VIII TWP. of S. NORWICH CO. of OXFORD ONTARIO		
PROJ. 1022	DATE JAN. 1963	DWG. No 1

12 x 24