

BA 1117  
WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS  
LABORATORY TESTING  
SOIL MECHANICS CONSULTATION

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



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

Project: J 571

September 5, 1960.

Mr. W. McDowell, P. E.,  
Asst. County Engineer,  
Norfolk County,  
c/o McDowell & Jewitt  
Consulting Engineers,  
92 Kent St. South,  
Simcoe, Ont.

Foundation Conditions  
Embankment Approaches, Big Creek Crossing

Dear Sirs:

This report supplements our preliminary letter of August 27th, which was written during the field investigation program of the site noted above. The information contained in the following paragraphs essentially confirms observations and comments made on that date, although our thoughts on pile refusal depths have been qualified somewhat.

Our observations and recommendations arising out of this investigation are presented below for your consideration. The factual information and reasoning, which form the basis for these conclusions, are considered in detail later in the report.

Observations and Recommendations

1) The soil below the surface of the Big Creek flood plain consists of approximately 20 feet of loose sandy alluvium and then by strata of medium dense silt and of stiff clay, each of which are about 10 feet thick. Very dense coarse silt underlies the clay at approximate Elev. 551 feet, or about 44 feet below the flood plain surface. The upper approximately 10 feet of alluvium is slightly cohesive in the vicinity of hole 1, but at other test locations relatively free draining sand exists. A stratum of stiff clay, overlain by a surface mantle of fine sand, was noted on the west slope.

2) Except for the slightly cohesive sand alluvium noted in the vicinity of Hole 1, all soil strata are quite competent to support the proposed embankment weights safely. If load is applied too rapidly on the upper cohesive sand, embankment failure may take place.

3) In order to minimize this danger, embankment fill should be placed across the valley floor first and then built up gradually along the west slope until final grade is reached. Big Creek, which passes near the centre line of the proposed embankment and adjacent to this cohesive soil, can be used to hasten the consolidation of this compressible alluvial material. In order to accomplish this, the creek channel should be cleared of debris and then filled with sand. As embankment load is applied, pore water will be pressed out of the upper 10 feet of soft alluvium and into this permeable channel. In this way, the soft soil will adjust quickly to the weight of the embankment.

4) On the basis of information obtained from holes 1 and 2, which are located at least 500 feet west of the bridge site, cylindrical piles will definitely encounter refusal near the top of the dense silt stratum which begins at Elev. 551 feet, or about 33 feet below creek bed level. The uncorrected penetration resistance of the upper few feet of this silt is in the order of 70 blows per foot. Experience in the Toronto area indicates that cylindrical steel piles cannot be driven very far into soil having a penetration resistance exceeding about 35 blows per foot.

However, according to local experience at other adjacent bridge sites over Big Creek, refusal to wood piles has been encountered at higher levels than this dense silt. Since a smaller driving energy is used for wood piles, it is conceded that similar conditions could occur at this crossing. The upper stratum of medium dense silt, beginning below Elev. 573 feet, has a penetration resistance of the order of 20 blows per foot. This material offered considerable resistance to the advancement of the casing and it was necessary to wash ahead in order to make reasonable progress. On the basis of this field information, we would consider the upper silt to be border line material as regards its ability to offer refusal to wood piles. In order to take advantage of this indefinite condition, it is recommended that the piles be paid for on a footage-in-place basis. In addition, the refusal condition of one or more test piles should be checked by attempting to re-drive them to greater depths a few hours after initial refusal has been encountered. A temporary high resistance to penetration can be experienced in silty soils because the pore water between the silt particles cannot drain away quickly enough.

5) It is proposed to use about 38 wood piles, placed within an area 10 feet wide by 30 feet long, to support the pier loads of this bridge. The desired load per pile is in the order of 20 tons. When driven within this small area, the pile group will tend to perform as a deep footing. Neglecting buoyancy effects, the net addition of stress to the soil at the pile tips will be about 5000 p.s.f. Assuming soil conditions similar to those indicated for hole 2, the estimated settlement under this application of load should be in the order of 2 inches. Although the abutment loads will not be so great, the addition of about 5 feet of fill adjacent to the bridge will cause them to settle about the same amount.

6) The change in flow pattern under the bridge, after the approach embankments have dammed back the flood waters, is a matter for hydraulic study. We would expect that the velocity of flow will be increased somewhat and, as a consequence, a greater danger of creek bed scour exists. Accordingly, we recommend that the river slopes in the vicinity of the bridge site be protected with riprap up to high water level. This riprap should be placed on a bed of well-graded gravel at least 12 inches thick. The gravel will serve as a filter to prevent the sucking-out of fines through the voids in the riprap as the river flows past.

The field observations and reasoning, which form the basis of the foregoing comments are considered under the following headings:

#### The Project

The county road between north and south Walsingham Townships presently crosses channels of Big Creek at two locations, as shown on the key plan in Dwg. 1. The river meanders across its flood plain which is about 1300 feet wide at this location. Both bridges are in poor condition and therefore it is proposed to replace them by a new structure, to be located near the east side of the flood plain. This bridge will be about 155 feet long and the river will be diverted through culverts under it.

The west approach to the bridge descends about 70 feet in elevation down the west boundary of the valley. The existing road down this slope is quite steep and therefore, it is proposed to ease the grade by the installation of higher embankments, as shown in Dwg. 1. The purpose of this investigation was to determine if these embankments would be stable.

#### Field Investigation and Subsoil Description

Two cased borings were made at this site near the west end of the flood plain, as shown on Dwg. 1. One of these, hole 1, was taken about 70 feet below ground surface, while hole 2 was terminated 55 feet below flood plain level. Since most of the soil encountered was granular, samples were recovered in a partially disturbed state using the conventional 2-inch O.D. split spoon. In each case, a driving energy of 350 ft.lbs. per blow was used to drive the sample into the ground below the bottom of the casing. This is the energy specified for the Standard Penetration Test.

Some 2-inch I.D. thin-walled Shelby tube samples were obtained in the deposit of clay noted below 34 feet and field vane tests were carried out between sampling intervals in this material. Dynamic cone penetration tests were performed adjacent to each hole in order to provide an additional check on the relative density of this soil.

Shallow auger borings or pits were made on the west slope in order to establish the contact between the sand and clay noted on this hillside and to determine the condition of the clay. The locations of this work are indicated on Dwg. 1.

The soils information, obtained from holes 1 and 2, is indicated on the borehole logs, presented as Dwgs. 2 and 3 of this report. Four distinct soil types are indicated.

The top stratum, below flood plain level, consists of an alluvial deposit which extends 20 to 22 feet below the ground surface, or to Elev. 573 feet. It exists in a loose condition and it is composed, for the most part, of fine sand with some organic debris. It is somewhat more plastic in the first approximately 10 feet of the hole 1 location. Field vane measurements, within this depth, indicated a shear strength ranging from 800 to 1200 p.s.f. This test measurement provides an approximate indication only, of the shear strength of this soil because it will tend to drain as shear stresses are imposed.

Below Elev. 573 feet, medium dense silt with some very thin layers of clay was encountered. This stratum is about 11 feet thick and it offered a penetration resistance of approximately 20 blows per foot to sampling equipment. It was somewhat difficult to drive casing into this material, and attempts were made to ease this difficulty by washing ahead with a large chopping bit.

Stiff reddish clay lies below this silt, between Elev. 562 and 551 feet. It contains fine sand partings at irregular intervals, ranging from about  $\frac{1}{4}$  to 2 inches apart. According to laboratory triaxial and field vane tests, its shear strength was at or in excess of 1850 p.s.f. Its moisture content was well below the liquid limit value of 42 percent.

Very dense coarse silt underlies the clay below Elev. 551 feet, or about 46 feet below the surface of the flood plain. The penetration resistance of the upper levels of this material is in the order of 70 blows per foot.

#### Embankment Stability

The specific purpose of this investigation was to determine whether it was safe to place fill down the west slope of Big Creek valley. In order to determine if the underlying soil is strong enough, two borings were made near the western edge of the flood plain where the embankment fill was greatest. The proposed embankment grade and the estimated subsoil stratigraphy under this westerly approach to the bridge are indicated in Dwg. 1.

Except for exceptional cases where failure has occurred in silty soils due to the introduction of heavy vibrations, instability of embankment foundations usually results from the presence of a weak stratum of cohesive soil.

Reference to the results of this boring program indicates that a more or less horizontal stratigraphy exists. Assuming that this arrangement of soil strata extends under the bridge site 500 feet to the east, it can be seen that the structure must be supported on some type of pile foundation. The upper flood plain deposits are too loose and too subject to river scour to be considered as a satisfactory permanent base for the bridge.

The first stratum of any competence is a layer of medium dense silt which begins about Elev. 573 feet. According to local experience gained at adjacent bridge sites, it is possible that wood piles have reached refusal in this material. The penetration resistance of this soil was found to be 20 blows per foot in hole 2, the closer boring to this site. Although a penetration resistance of this magnitude is not sufficient to provide refusal to cylindrical steel piles, it is possible that wood piles may be stopped after a few feet of penetration into this material. If refusal is encountered, attempts should be made to re-drive some of the piles. This is to ensure that the high resistance is not a temporary condition resulting from the low permeability of the silt. If refusal is not experienced in this upper silt, it should be met in the very dense silt stratum which begins at Elev. 551 feet, or about 33 feet below creek bed level.

It is understood that each pier of the bridge will be supported on about 38 wood piles, each of which will be loaded to approximately 20 tons. The perimeter of this pile group has been given as approximately 10 by 30 feet. When placed in this confined area, the pile group will tend to act as a deep footing. The bearing stress at the pile tips will be in the order of 5000 p.s.f.

If it is assumed that the piles are stopped at a depth of 25 feet in the upper silt, or at Elev. 570, they will bear 14 feet above the mid-depth of the stiff clay. Assuming a 30 degree load spread below the pile tips, the addition of stress to the mid-point of the clay will be about 1300 p.s.f. Since the clay is highly overconsolidated, this addition of stress will produce a slight recompression of the clay, only. According to experience obtained on other similarly over-consolidated materials, the modulus of compressibility,  $M_v$ , of this soil is probably in the order of 0.006 sq.ft. per kip. The amount of settlement can be estimated from the expression:

$$S = H M_v p$$

where  $H$  is the thickness of the clay stratum = 120 ins.  
 $p$  = 1300 p.s.f.

The estimated settlement, here, for the values given, is about 1 inch. In order to allow for compression of the overlying medium dense silt, this value should be increased to 2 inches. A detailed settlement analysis, involving the performance of consolidation tests on samples taken from the bridge site area, would be required for a more accurate estimate of settlement.

It is understood that the abutment loading will not be as great as is indicated for the piers. However, this load unbalance should be compensated for by the surcharge effect of about 5 feet of fill of the embankment approaches. Therefore, within the accuracy of estimation possible with present information differential settlement should not be a problem.

If possible, piles for the bridge should be driven before embankment fill is placed adjacent to it. This is suggested as a precaution against local embankment failure into the creek, which could be precipitated by vibrations set up in the sand alluvium during pile driving operations. Alternatively, the creek bed, adjacent to the structure, could be filled with sand during bridge construction, and then removed when this work has been completed.

We hope that the comments of this report are of assistance to you in the design and construction of this river valley crossing. If we can be of any further assistance, we shall be pleased to hear from you.

Yours very truly,

*W A Trow*




William A. Trow (P. Eng.)

WAT/lt  
Encls.







## 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  151 +

## 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

X<sup>LI</sup>

—○—

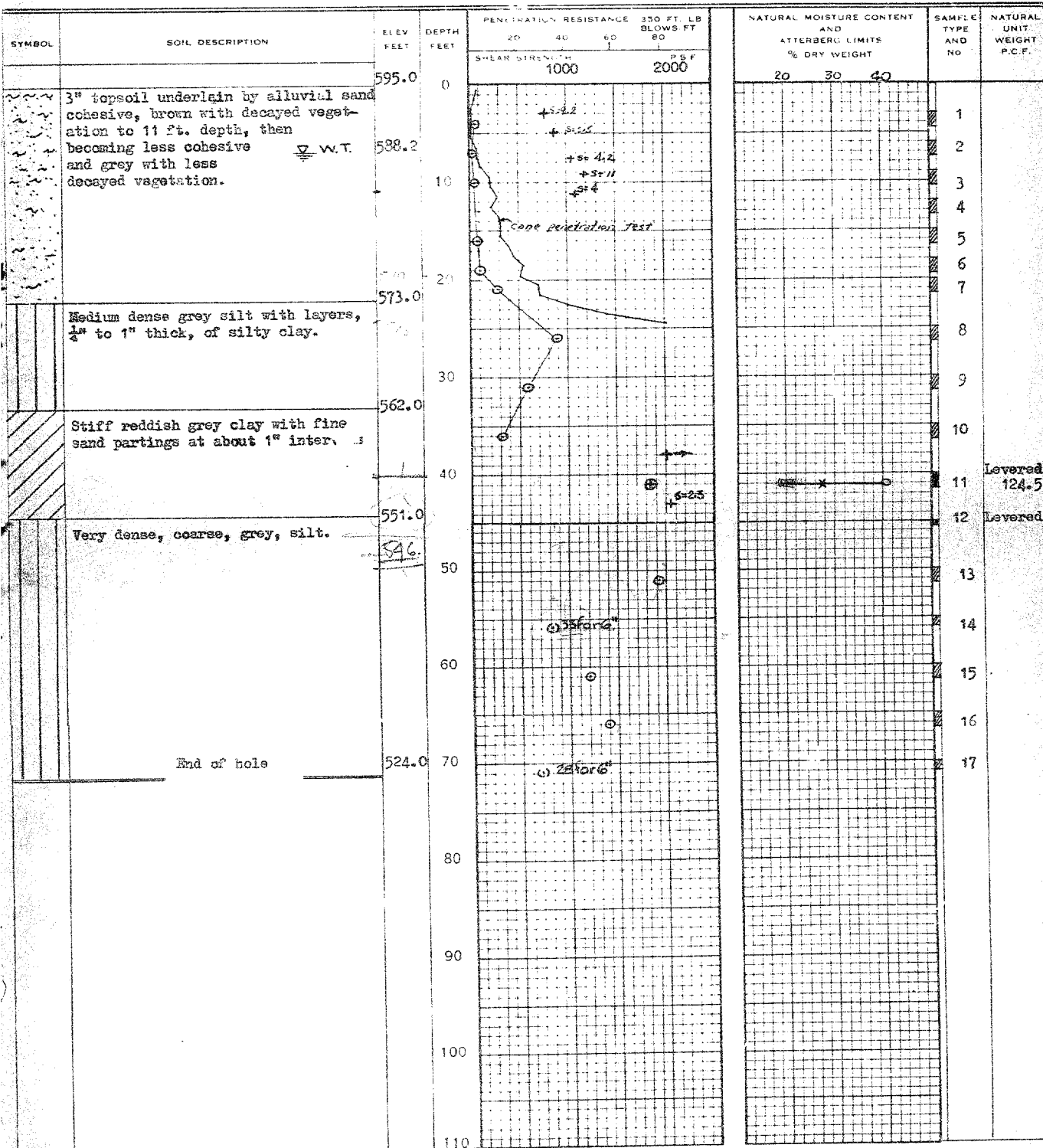
—|—

—■—

—■—




—■—

BOREHOLE NO. 1  
 PROJECT Norfolk County Bridge  
 LOCATION South-West of Simcoe  
 HOLE LOCATION See Dwg. 1  
 HOLE ELEVATION 595.0  
 DATUM Geodetic






## LEGEND

## PENETRATION RESISTANCE



2" O.D. SPT. TUBE   
 2" O.D. SHELBY TUBE   
 2" DIA. CONE 

## SHEAR STRENGTH




UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE   
 UNCONFINED COMPRESSION   
 VANE TEST AND SENSITIVITY  $15, \pm 5$  

## 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 

LI

X

—○—

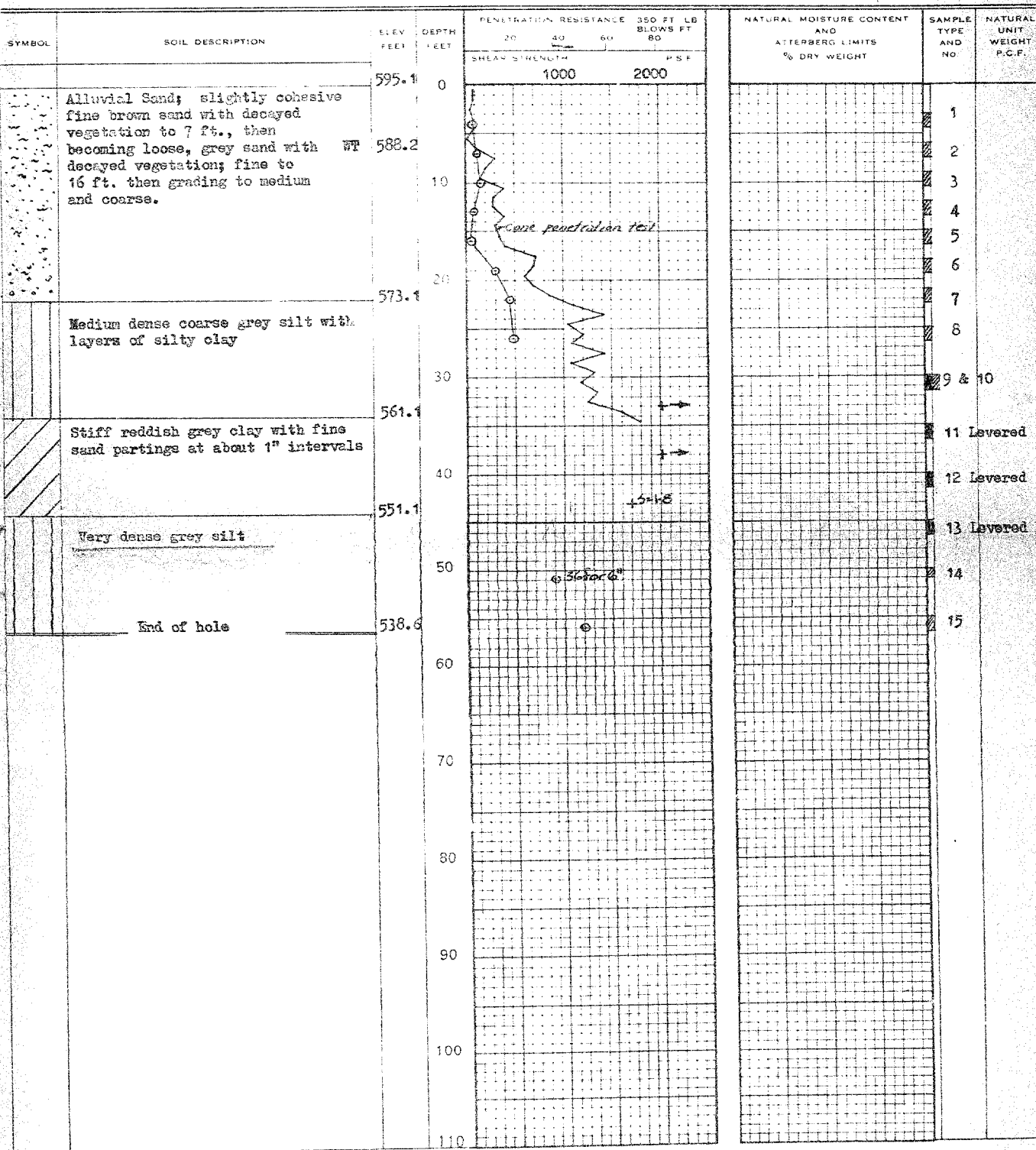
—

■

■

■

BOREHOLE NO. 2  
 PROJECT Norfolk County Bridge  
 LOCATION South-West of Simcoe  
 HOLE LOCATION See Map.  
 HOLE ELEVATION 595.1  
 DATUM Geodetic





G.I.-30 SEPT. 1976

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 40 I10-6

W.P. No. \_\_\_\_\_

CONT. No. \_\_\_\_\_

W. O. No. \_\_\_\_\_

STR. SITE No. \_\_\_\_\_

HWY. No. \_\_\_\_\_

LOCATION EMBANKMENT  
APPROACHES, BIG CREEK

=====

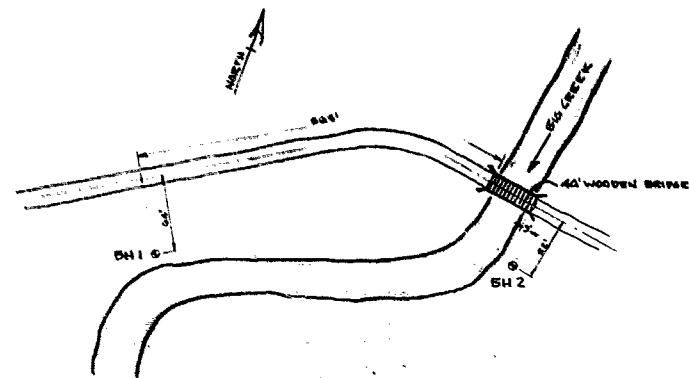
OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. ONE

REMARKS: \_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_



BOREHOLE LOCATION SKETCH  
SCALE 1"=60'

