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B R I D G E

H O R N E R ' S C R E E K

E A S T Z O R R A T W P .



0-7284

April 1965

William Trow Associates Limited

Project: J1904

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Associates Ltd.

County of Oxford,
Court House,
P.O. Box 397,
Woodstock, Ontario.

April 9, 1965

Attention: Mr. J. N. Meathrell, P. Eng.
County Engineer

Foundation Investigation
Proposed Bridge Replacement, Horner's Creek
East Zorra Township, County of Oxford, Ontario

Dear Sirs:

In conformance with your authorization, we have completed a foundation investigation at the above site. The field work associated with this project was carried out March 31, 1965.

SUMMARY

- 1) The subsoil at this site consists of a dense silt till. This till contains considerable quantities of sand and some clay. An alluvial deposit and road fill overlies this material.
- 2) Foundations for the proposed structure should be designed using simple spread footings and a safe net bearing pressure of 4 tsf. Scour protection must be provided and these footings shall not be placed above El. 86.0 ft.
- 3) No problems associated with settlement, approach stability or construction of footings are envisaged at this site.



THE SITE

The site of the proposed bridge replacement is located just north of Highway No. 97 between the Villages of Cassel and Ratho. At the site, Horner's Creek flows in a southerly direction. The water depth near the bridge was a maximum of 2 feet deep, while marks on the bridge abutments indicate that levels 2 feet above present water level are common. The stream bed consists of boulders and sand and gravel over black peaty clay. A thin steel rod penetrated a maximum of 10 inches through the stream bed material, under the weight of one man.

The existing steel bridge is supported by concrete abutments. These abutments are in relatively poor condition with various vertical cracks 1 inch wide visible on the north pier.

FIELD WORK AND SUBSOIL STRATIGRAPHY

The field work at this site consisted of two sampled boreholes located as shown on Dwg. 1. The boreholes were advanced uncased using continuous flight auger equipment. Samples of the subsoil were obtained using conventional split spoon or Shelby type samplers.

Detailed information as to the subsoil encountered in the boreholes has been included, along with the results of laboratory testing, on the borehole logs Dwgs. 2 and 3. An estimated subsoil stratigraphy is shown on Dwg. 1. Borehole elevations are relative to the temporary bench mark as shown on Dwg. 1.



The subsoil at this site essentially consists of a dense to very dense silt till. This silt till contains more sand in the upper regions than is found at depth. At a depth of 25 - 30 feet, considerable clay was noted. Overlying the silt till a heterogeneous alluvial deposit, thought to be a result of stream deposits, was proven. Road fill was also encountered in the boreholes as the holes were placed through the road surface.

FOUNDATIONS

Foundations for the proposed structure may be designed as simple spread footings founded at least 4 feet below maximum river scour depth. Examination of the borehole logs and stream bed indicates that the maximum depth of scour is near the present stream bed level, i.e. El. \pm 90.0 ft. Therefore spread footings should be founded at or below El. 86.0 ft. These footings can be designed using a safe net bearing pressure of 4 tsf. In addition to the scour protection achieved by placing the footings at El. \pm 86.0 ft, both river banks up and down stream of the bridge should be provided with rip-rap protection.

The settlement of spread footings designed using the suggested bearing pressures will consist of a negligible amount of elastic settlement of the subsoil. This settlement will not exceed 1/2 inch.



Construction of the footings will involve excavation below the groundwater table. Some problems associated with the entry of groundwater into the excavations must be expected. The major portion of this water will drain through the alluvial material. To facilitate construction, two methods of controlling the groundwater are suggested:

a) Sheeting - Interlocking steel sheeting driven to Bl. \pm 80 ft. or a minimum of 6 feet beneath the base of the excavation will provide the most positive control of ground seepage water. If this sheeting is left in place it will provide additional scour protection.

b) River Diversion - It will be possible to dewater the excavation by pumping if the river is directed away from the abutment and retained by a temporary earth dam. A large excavation - larger than the size of the proposed footing, - will be required to accommodate some sloughing of the side walls and shallow temporary drains at the base of the excavation. The base of such an excavation - based on the high density and visual examination of the samples - will remain stable.

APPROACH FILLS

Because of the dense nature of the subsoil, no problems associated with the stability of the approach fills are present at this location; however the 2 - 4 feet of organic alluvium near the river banks should be removed from the area between the wing walls prior to placing the embankment. The approaches should incorporate 2:1 side slopes and standard construction procedures.

EARTH PRESSURES

For closed abutments, the horizontal earth pressures exerted against them must be considered. It is suggested, for a simply supported structure, that the earth pressure coefficient to be used in calculations equal 0.25. This value assumes a slight yield of the abutment if compaction is such that the earth pressure tends to rise above 0.25 and approach the 'at rest' condition. This slight yield will then reduce the earth pressure coefficient to 0.25. With adequate drainage facilities behind the retaining abutment, the earth pressure p at any given depth h can be determined from the expression:

$$p = 0.25 \gamma h + 0.25q$$

where: γ = 130 pcf the estimated unit weight of backfill material

q = the value of any surcharge (in psf) acting near the abutment

It is hoped that these comments will assist in the design of the foundations for the structure. If we can be of further assistance, please do not hesitate to contact this office.

Yours very truly,

K. Peaker.

K. Peaker, P. Eng.

KP/chm
Encls.

WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 2
PROJECT No. 1904

LEGEND

BOREHOLE No. 1
PROJECT Proposed Bridge Replacement
LOCATION East Zorra
HOLE LOCATION See Dwg. 1
HOLE ELEVATION 99.82 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) +^s

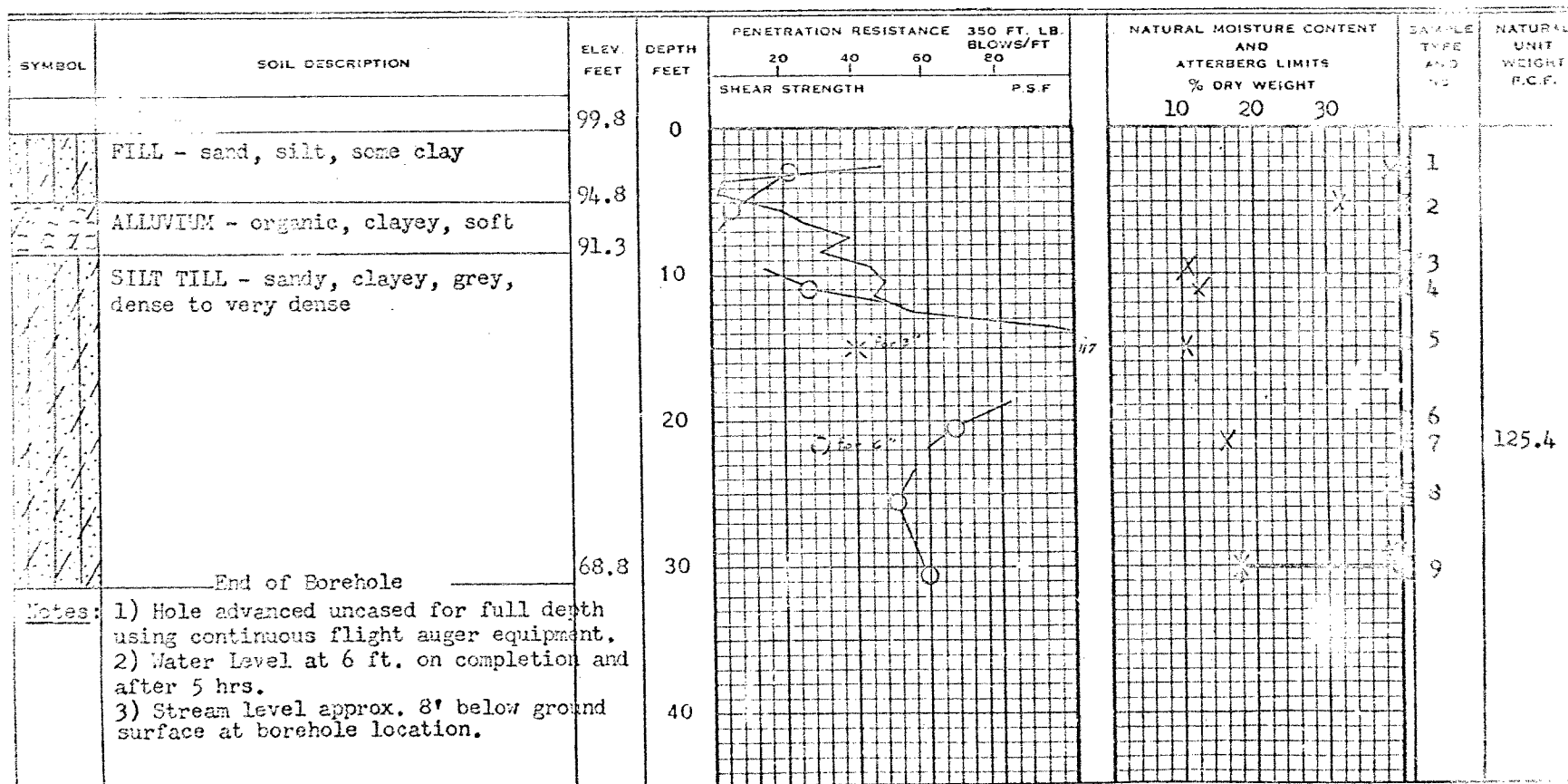
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

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. 11901

BOREHOLE NO. 2
PROJECT Proposed Bridge Replacement
LOCATION East Lerma
HOLE LOCATION See Dwg. 1
HOLE ELEVATION 100.02 ft.
DATUM See Dwg. 1

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—
2" I.D. SHELBY TUBE —X—X—X—X—
2" DIA. CONE —————
SHEAR STRENGTH —————
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) †

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

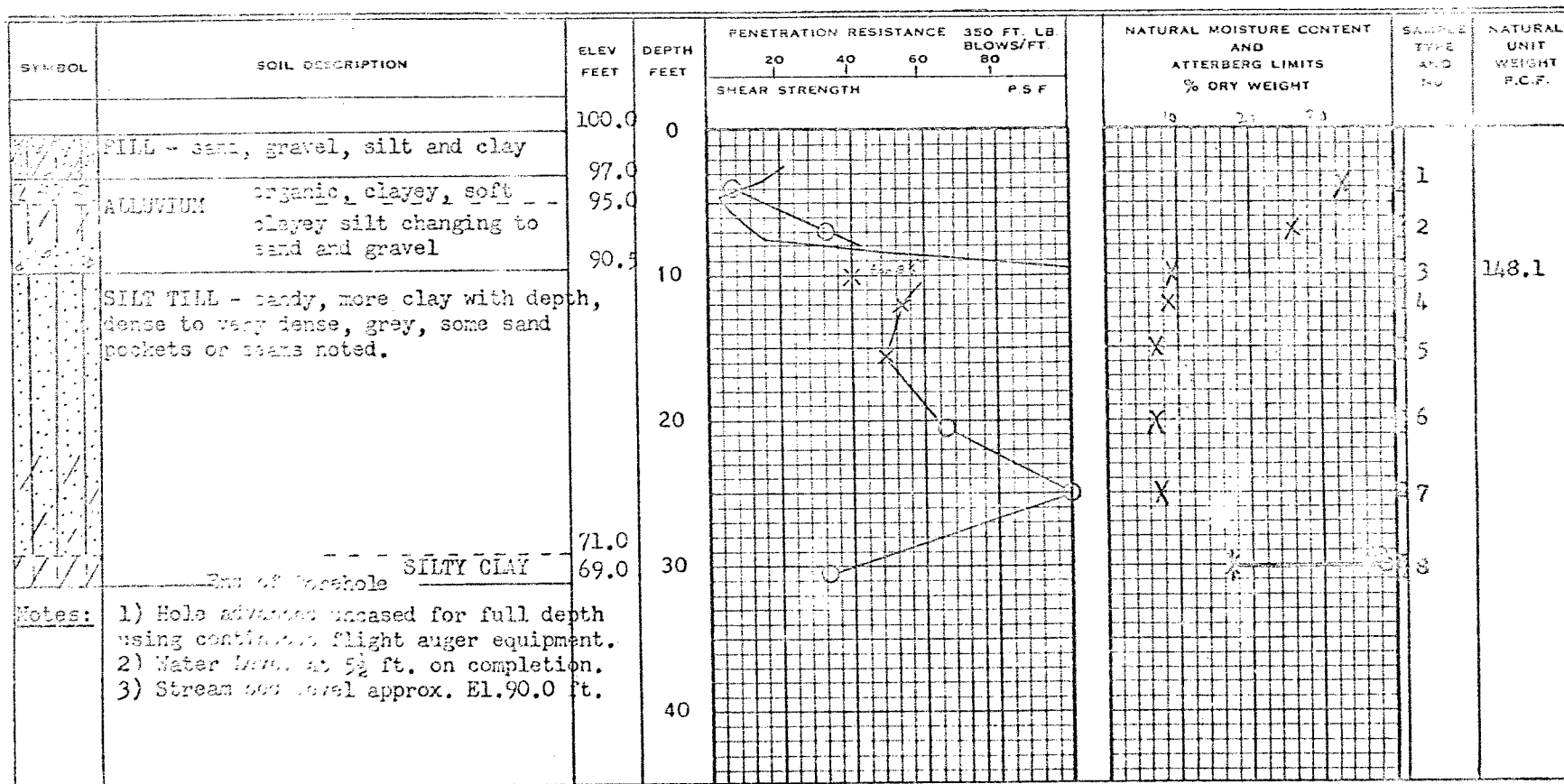
ATTERBERG LIMITS

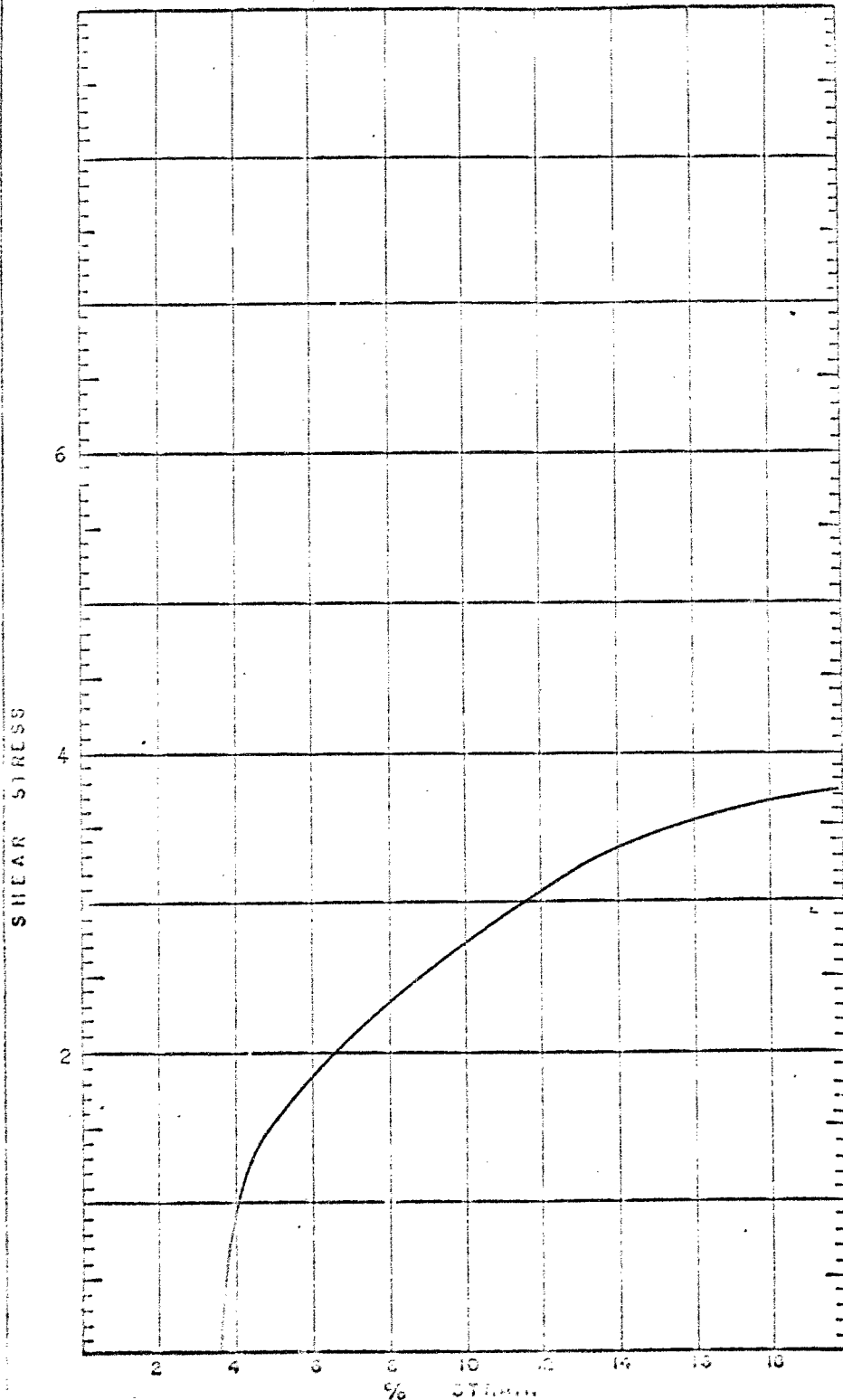
LIQUID LIMIT ———○———

PLASTIC LIMIT ———|———

SAMPLE TYPE

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





TEST NO

TEST

S.H.

C

X

W

 σ_3

SOIL

TEST NO

TEST

S.H.

C

X

W

 σ_3

SOIL

TEST NO

TEST -Cycled Undrained

S.H. 2

C 3700 psf.

X 148.1 pcf.

W

 σ_3 9.8 psi.SOIL Salt till
with some gravel
sizes.

TRIAxIAL TEST RESULTS

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT