

DOCUMENT MICROFILMING IDENTIFICATION.

GEOCRES No. 30M 13-42

DIST. 6 REGION CENTRAL

W.P. No. _____

CONT. No. _____

W. O. No. _____

STR. SITE No. _____

HWY. No. _____

LOCATION ELLIS BRIDGE,

~~NEAR~~ HUNTER RIVER

WHITE SCHOOLHOUSE ROAD

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____

REMARKS: DOCUMENTS TO BE UNFOLDED BEFORE

MICROFILMED

G.I.-30 SEPT. 1976

B.A. 2040 ↓

DURCAN HOPPER ASSOCIATES LIMITED
1885 WILSON AVENUE
WESTON, ONTARIO



RECEIVED
MAR 29 1965
A. D. D.
G. K. H.
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R. C.
P. F.
J. G.

FOUNDATION INVESTIGATION
PROPOSED RECONSTRUCTION ELLIS BRIDGE
TOWNSHIP OF VAUGHAN
WHITE SCHOOLHOUSE ROAD

Project: J1873

March, 1965

William Trow and Associates Ltd.

80 Milner Drive
Weston, Ontario
749-1290

William Trow

Project: J1873

Soil Mechanics
Consultants
W. A. Trow
MSc. MEIC. P. Eng.
K. Peaker
PhD. MEIC. P. Eng.
G. H. Shields
PhD. MEIC. P. Eng.



Associates Ltd.

Duncan Hopper Associates Limited,
1085 Wilson Avenue,
Weston, Ontario.

March 26, 1965

Attention: Mr. R. Crawford, P.Eng.

Foundation Investigation
Proposed Reconstruction Ellis Bridge
Township of Vaughan
White Schoolhouse Road

Dear Sirs:

In conformance with your authorization dated March 8th, 1965, we have completed a foundation study at the above site. Our findings and recommendations are summarized as follows:

1) The site of the proposed reconstruction is underlain by a shallow deposit of sandy silt clay alluvium followed by a deep deposit of silt or silt till.

2) Foundations for the proposed bridge are recommended to be of the spread footing type designed for a safe net bearing value of 4 tsf. These footings should be founded at or below $El \pm 651$ for scour protection, or alternatively at $El \pm 655$ if steel sheeting is used as scour protection.



3) Alternative piled foundations for spill-through type abutments have been discussed. These piles, of the cylindrical displacement type, can carry a minimum safe load of 50 tons/pile if driven to the elevations suggested in the body of the report.

4) Comments related to the stability of excavations below groundwater level have been given in the report. No embankment stability problems exist at this site. Earth pressure coefficients for the design of earth pressures on abutments have been suggested.

These comments have been reached after consideration of the following details:

THE SITE

The proposed Ellis Bridge crossing a branch of the ~~San~~ ^{San Joaquin} River is located approximately $\frac{1}{2}$ mile east of highway No. 27 along the White Schoolhouse Road. At present a concrete arch type structure of 50 feet span bridges the river. This existing structure built about 1922 appears to be in good condition. The river at the site flows from north to south. At the time of the field investigation (March 16th, 1965) the river was 2.2 feet above the zero datum mark on the water level gauge board.

The existing bridge will be replaced by a wider structure with approximately 70 feet span. The existing road centreline will remain unchanged at this site.



FIELD WORK AND SUBSOIL STRATIGRAPHY

Three sampled borings comprise the field work at this site. These borings were supplemented by dynamic cone tests adjacent to boreholes 1 and 2. Boreholes were advanced using uncased boreholes and conventional continuous flight auger equipment. Samples of the subsoil were obtained using standard split spoon and Shelby type samplers.

The subsoil encountered is shown in detail on the borehole logs Dwg. 2 to 4, while an estimated subsoil stratigraphy is included with the borehole locations, on the site plan drawing. All boreholes have been referenced to the bench mark as indicated on the site plan drawing, the elevation of which was provided by Duncan Hopper and Associates Limited.

The subsoil at this site consists essentially of an upper deposit of clay, sand and silt, which may have resulted from earlier alluvial deposits underlain by a dense silt or silt till. The upper deposit of so called fill material is loose, the lower portions contain considerable quantities of sand, probably the old stream bed. The density of the silt or silt till stratum was established by the penetration resistance of the split spoon sampler and the dynamic cone.

FOUNDATIONS

The foundations for the proposed structure are recommended to consist of simple spread footings. These footings must be founded approximately 4 feet below



river scour level. On the basis of the borehole information the maximum depth of scour at this site is estimated to be to El \pm 655, thus all footings should be placed at or below El \pm 651. Based on the estimated density of the till material these footings can be designed using a safe net bearing value of 4 tsf.

An alternative to placing the footings at El \pm 651 is to provide scour protection using steel sheeting and to raise the footing level to El \pm 655. If this solution is adopted steel sheeting should be driven on the three exposed sides of the abutment to El \pm 645. The suggested bearing value of 4 tsf may also be used at this elevation. Whether the footings are placed at depth, or steel sheeting is utilized to protect the footings, both the upstream and downstream slopes adjacent to the abutments should be protected by suitable rip rap.

Using the suggested bearing pressures the settlement of the structure will be well within the tolerable limits of a single span structure, (total settlements should not exceed 1 inch). Because of the granular nature of this subsoil most of the settlement will be complete at the end of construction.

It can be argued that a piled foundation is suitable at this site if a spill-through type abutment is utilized. With this method a pile cap placed near the top of the embankment would be supported by piles driven into the silt till. It is anticipated that cylindrical piles will meet refusal near El \pm 648; however, some piles may continue beyond this depth.



If piles penetrate beyond El \pm 648 they should be driven to El \pm 635 or to refusal whichever occurs first. The piles can be designed to carry a load as a short column if they meet refusal, or can be designed for a safe load of 50 tons/pile, provided they reach refusal or are driven to El 635. Confirmation of the pile capacity can be obtained from the relationship:

$$Q = A\gamma DN + \frac{pk \gamma D^2}{2}$$

where:

- A is the area of the pile tip = 0.78 sq. ft. for a 1 ft. diameter pile
- γ is the unit weight of the sand estimated to be at least 127 pcf above the water table and 65 pcf below
- D is the depth from the base of the pile cap in the compacted fill to the tip of the pile. Considering that the fill will be compacted and the underlying soil will adjust immediately to its weight this length of D is believed to be reasonable
- N is a bearing capacity factor estimated to be equal at least to 300* for the bearing depth and soil density applicable
- k is the resultant friction coefficient on the shaft estimated to be equal to 1.0 below El \pm 656
- p is the perimeter of the pile = π for 12 inch diameter pile

Solving Q ultimate = 265 Tons approximately for a depth of penetration equal to 20 feet into the silt till. This will provide an adequate safety factor for piles designed to carry a safe load of 50 tons/piles.

* G.G. Meyerhof - Some Research on the Bearing Capacity of Foundations - Canadian Geotechnical Journal, 1963.



FOUNDATION EXCAVATIONS

If the footing proposal is used, foundation excavations below the water table in porous sand and silt will be necessary. To permit the removal of water, the base of the excavation should be crowned with the footing area in the highest part of the cut and with the ground sloped down beyond the edge of the footing at 3 horizontal to 1 vertical. Seepage will tend to drain to the low edges of the excavation from which it can be removed by pumping. To avoid base heave caused by layering of more permeable material holes augered 10 to 15 feet below the base and backfilled with gravel should be installed. These 'wells' will relieve any pressures which tend to build up beneath the base of the excavation.

APPROACH EMBANKMENTS

No problems associated with the stability of approach embankments exists at this site.

EARTH PRESSURES

If abutments and wing walls are used on this project, i.e. the approach fill does not spill through the abutments, they must be designed to withstand the lateral earth pressure exerted by the retained soils. The earth pressure that will act on the walls can be estimated using a value of earth pressure coefficient equal to 0.35. The earth pressure, p , on the walls at any depth, h , can be found from the expression:



$$P = K \{ \gamma (h - h_r) + \gamma_s h_r + q \}$$

where:

- K = 0.35, the recommended earth pressure coefficient assuming the walls to be rigid. This value can be reduced to 0.25 if a slight inward movement of the abutment is possible
- γ = 125 pcf, the estimated unit weight of the retained soil
- γ_s = 60 pcf, the estimated submerged weight of the retained soil
- h_r = height of water table above the point being considered
- q = surcharge, if any, acting at the top of the wall.

The stability of the abutment and wing walls should be checked for horizontal sliding along the footing base. The resistance against the sliding is the frictional force acting along the footing base. The frictional force developed along the footing base can be calculated using a friction coefficient of 0.7 (concrete sliding on granular soils).

If the resisting force is less than $1\frac{1}{2}$ times the estimated sliding force, the footing base can be extended under the fill to increase the weight of backfill carried by it, or keyed into the subsoil. In this manner, the resistance to sliding can be increased.



Should any queries arise concerning the contents
of this report we will be pleased to discuss them with you.

Yours very truly,

K. Peaker.

K. Peaker, P.Eng.

KP/ss
Encls.

(2)
J1873



Existing Bridge
And Road
Looking East



Bridge Site
Looking North

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 ————

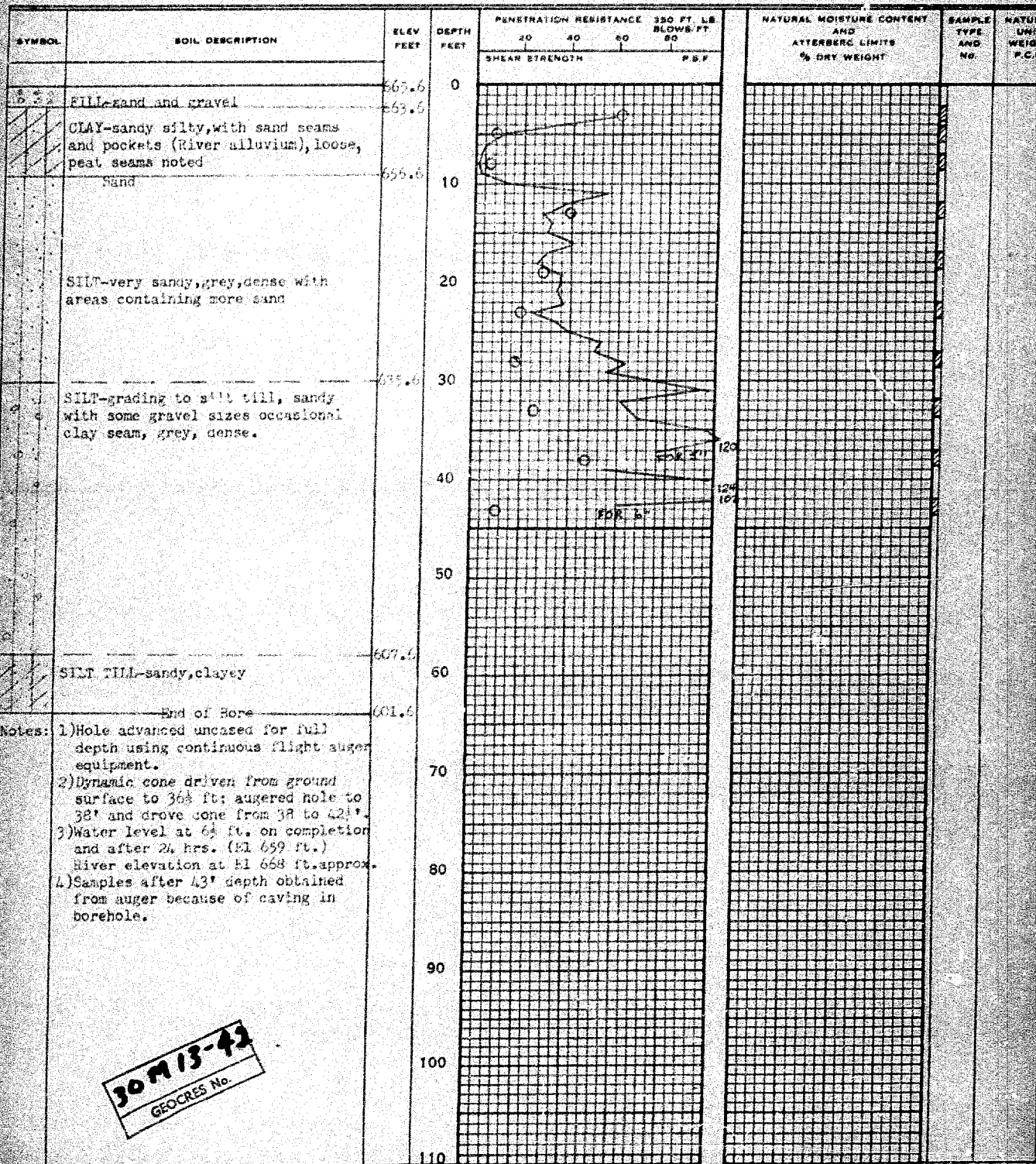
SAMPLE TYPE

2" O.D. SPLIT TUBE —■—

2" I.D. SHELBY TUBE —■—

3" O.D. SHELBY TUBE —■—

BOREHOLE NO. 1
 PROJECT Ellis Bridge
 LOCATION White Schoolhouse Road
 HOLE LOCATION See Dwg. 1
 HOLE ELEVATION 665.65
 DATUM See Dwg. 1



WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING NO. 3
PROJECT NO. 1187

LEGEND

BORERHOLE NO. 2
PROJECT Ellis Bridge
LOCATION White Schoolhouse Road
HOLE LOCATION See Dwg. 1
HOLE ELEVATION 65.21 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 (91)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

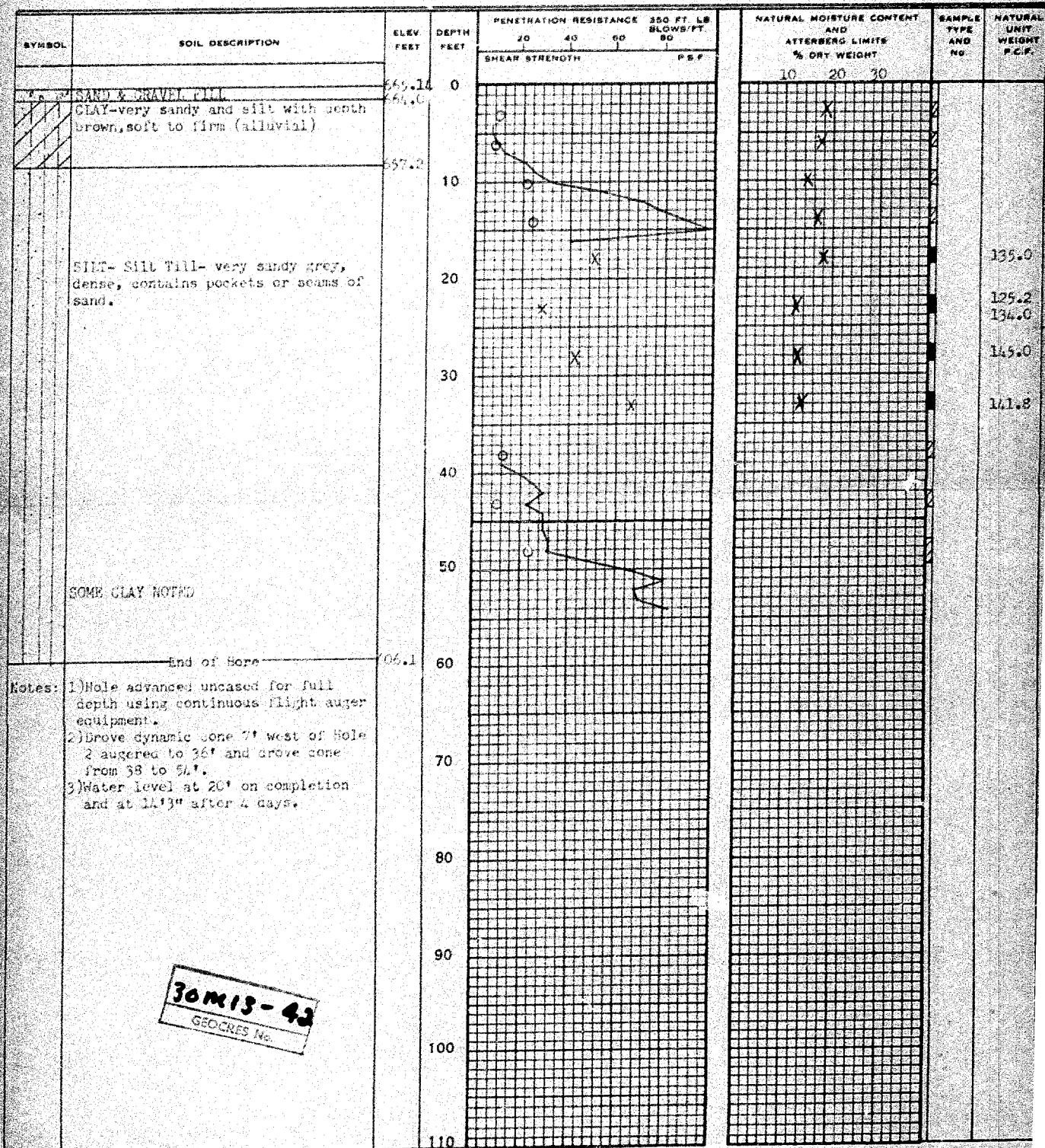
ATTERBERG LIMITS

LIQUID LIMIT

PLASTIC LIMIT

SAMPLE TYPE

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



30M13-42
GEOCRENS No.




SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

PROJECT NO. J1873

LEGEND

BOREHOLE NO. 3
PROJECT Ellis Bridge
LOCATION White Schoolhouse Road
HOLE LOCATION See Dwg. 1
HOLE ELEVATION 663.04 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 (2) +

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS

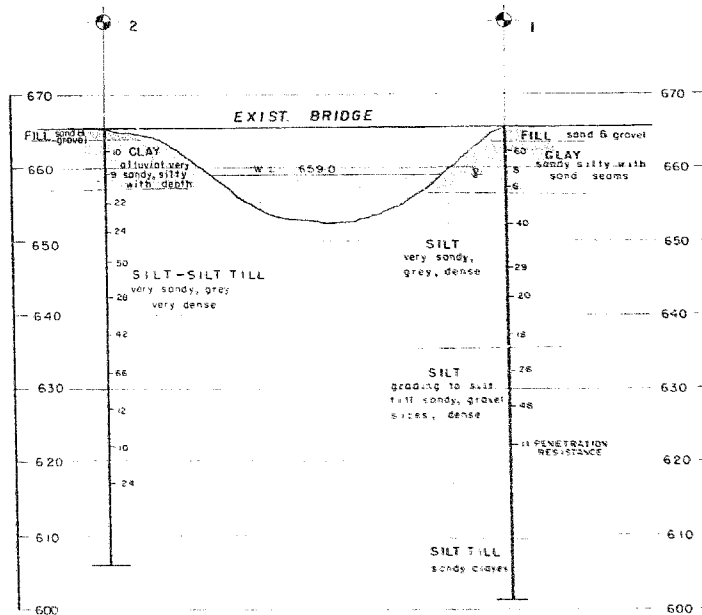
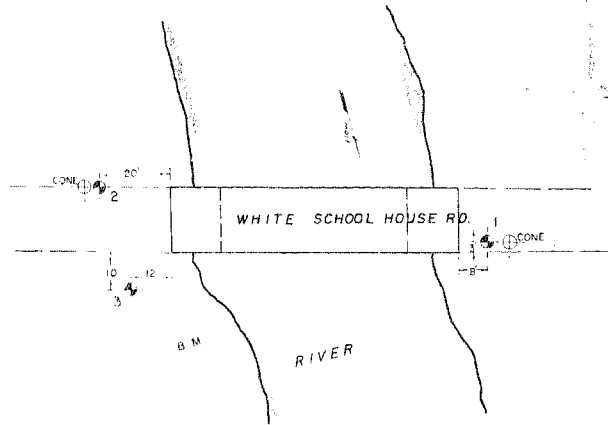
LIQUID LIMIT

PLASTIC LIMIT

SAMPLE TYPE

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

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	PENETRATION RESISTANCE				350 FT. LB. BLOW/FT. 20	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.
				20	40	60	80				
	5 inches Topsoil	663	0								
	CLAY-changing to silt & sand in lower regions layered (alluvial)	656.7									
	SILT-Silt Till - very sandy, grey, dense, contains pockets or seams of sand	658	10		X						
			20		X						
			30								
	End of Bore	627.5	40								
Notes:	1) Hole advanced uncased for full depth using continuous flight auger equipment. 2) Water level 6'3" on completion and after 4 days.										



INTERPRETED SUBSOIL STRATIGRAPHY

SCALE HOR. 1 IN. = 20 FT
VERT. 1 IN. = 10 FT

LEGEND

BOREHOLES

CONE

BENCH MARK EL. 662.74

Nail on north side of tree

30m13-42

- NOTE -

Samples will be kept for 3 months from the date of this report unless otherwise directed.

- NOTE -

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

William Trow & Associates Ltd.

FOUNDATION INVESTIGATION

PROPOSED ELLIS BRIDGE

WHITE SCHOOL HOUSE ROAD

TWP. OF VAUGHAN

ONTARIO

PROJ. 1873 DATE MARCH 1965 DWG. No. 1