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Golder Associates

REPORT ON

FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 10 TPM
FROM 0.3 km SOUTH OF HIGHWAY 9 NORTHERLY
1.2 km TO 0.3 km NORTH OF BROADWAY STREET
DISTRICT 33; SOUTHWESTERN REGION
GWP: 449-97-00

Submitted to:

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PART A – FIELD INVESTIGATION

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1.0 INTRODUCTION

Golder Associates Ltd. has been retained by Morrison Hershfield Limited (Morrison Hershfield) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation for the widening / improvement project for 1.2 km of Highway 10 in Orangeville, Ontario. The project includes extension of the proposed twin culvert carrying Broadway Avenue over Credit River and overhead sign installation on Highway 10. This report addresses the proposed twin culvert extension and three overhead signs within the project limits.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed culvert extension and overhead signs by drilling boreholes, and carrying out in-situ tests and laboratory tests on selected samples. Based on our interpretation of the data obtained, recommendations on the foundation aspects of design of the proposed works are provided. Comments are also provided on anticipated construction problems where they may affect the design of the proposed culvert extension.

The terms of reference for the scope of work are outlined in our proposal letter P91-8078, dated August 1999. The work was carried out in accordance with our Quality Control Plan for Foundation Design Services, dated September 21, 1999.

2.0 SITE DESCRIPTION

The site is located at the Broadway Avenue crossing of Credit River and on Highway 10 from about Broadway Avenue south approximately 400 m in the Town of Orangeville, Dufferin County.

The topography within the study limits is generally relatively level but with a 20 m differential in ground surface between the southern limit near Highway 9 and the northern limit north of Broadway Avenue. The ground slopes downward toward the north from about Elevation 426 m at the southernmost borehole to about Elevation 408 m at the Credit River crossing. The existing highway extends through residential, parkland and agricultural lands. Within the project limits, the vegetation cover generally consists of grass, bushes, and mature trees. The Credit River winds through a relatively wide floodplain area, which is heavily grassed.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out on November 4 and 5, 1999, at which time five (5) boreholes were put down at the site. Boreholes 1 and 5 were put down adjacent to the existing Credit River culvert – one on each side of Broadway Avenue. The borehole on the south side (Borehole 5) was put down through the road embankment since access to the floodplain area was not possible. The borehole on the north side (Borehole 1) was put down within the floodplain area in general proximity to the originally proposed extension limits. Boreholes 2, 3 and 4 were put at specified locations along Highway 10. A sixth hole (Borehole 4A) was put down at the north limit to provide information on the subexcavation under the existing Highway 10 embankment.

The investigation was carried out using CME 45 and 75 drill rigs supplied and operated by Lantech Drilling Contractors. In the boreholes, samples of the overburden were obtained at regular intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedures. The boreholes were extended to depths of between 5 m and 7.3 m below the existing ground surface. Groundwater conditions in the open boreholes were observed throughout the drilling operations. A standpipe / piezometer was installed in one of the boreholes to permit monitoring of the groundwater level at the site.

The field work was supervised on a full-time basis by a member of our technical staff who located the boreholes in the field, directed the drilling, sampling and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labeled containers and transported back to our laboratory in Mississauga for further examination. Index and classification tests were carried out on selected samples. The results of the testing are shown on the attached Record of Borehole sheets and on Figures 1 to 3.

The borehole locations were surveyed and staked in the field by Morrison Hershfield. The northing and easting co-ordinates and elevations of the borehole are indicated on the Record of Borehole sheets. The locations of the boreholes are shown on Drawing 1.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

The site is located at the intersection of two physiographic regions known as the Horseshoe Moraines and the Hillsburgh Sandhills (Chapman and Putnam, "The Physiography of Southern Ontario", 3rd Edition, 1984). The topography within these two regions is generally hilly, with knob and basin relief that is typical for moraines. The subsoils for this region are generally comprised of sandy materials, which are interlayered with glacial till. Interbeds of fine sand, silt, and clay are also common. Bedrock is generally deep below the ground surface in this region and at depths of greater than 50 m below the ground surface.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets following the text of this report. The stratigraphic boundaries shown on the borehole sheets are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoils along Highway 10 to the south of Broadway Avenue consist of surficial fills underlain by silty clay till which is in turn underlain by a silt deposit. At and north of Broadway Avenue, the subsoils consist of embankment fills or peat deposits underlain by a granular deposit comprised of sand to sand and gravel. The interpreted stratigraphic section at the Credit River culvert is shown on the attached Drawing 1. A detailed description of the subsurface conditions encountered in the boreholes for this investigation is provided in the following sections.

4.2.1 Pavement Structure and Fill

The pavement structure thickness as encountered in Boreholes 2 and 4 was 260 mm to 300 mm of asphalt underlain by 150 mm of Granular A base course. About 240 mm of Granular B was encountered under the Granular A in one of the boreholes. The pavement structure in Borehole 4 and the ground surface in Boreholes 3 and 5 are underlain by an essentially granular fill material.

The fill consists generally of sand and gravel with trace to some silt and occasional pockets of silt / clayey silt. The measured Standard Penetration 'N'values within the fill ranged from 4 blows to 49 blows per 0.3 m of penetration indicating a loose to dense state of packing. Generally the measured 'N' values are less than 20 blows per 0.3 m of penetration indicative of a compact state of packing.

At the Credit River culvert location, the road embankment fill on the south side of the road was found to extend to about Elevation 406.2 m and was directly underlain by a sand deposit. The organic deposit encountered at ground surface on the north side of the road was found to extend to about Elevation 406.7 m and was underlain by a sand deposit. At the location of Borehole 4A, the fill extended to 5 m depth which is greater than the height of the embankment. This information indicates that the organic deposits were probably removed from under the road embankment during construction.

4.2.2 Organic Deposits

About 1.7 m of peat and organic silt was encountered at ground surface in Borehole 1 located on the north side of the road at the Credit River culvert location. The organic deposits extend to about Elevation 406.7 m. One Standard Penetration Test within the peat gave an 'N' value of 4 blows per 0.3 m of penetration indicating a soft consistency.

4.2.3 Silty Sand to Sand and Gravel

In Boreholes 1, 4 and 5, a granular deposit consisting of silty sand to sand and gravel underlies the fill and / or organic deposit. The sand deposit directly underlying the organic and fill deposits is generally comprised of fine to coarse sand containing trace to some silt and gravel but ranges in composition to silty sand. The measured Standard Penetration 'N'values within the sand layer ranged from 5 blows to 17 blows per 0.3 m of penetration. The grain size distributions of two samples of the sand to silty sand deposit are shown on Figure 1.

These sands are extremely susceptible to piping / loosening where excavation is carried out through these deposits below the groundwater level.

The sand layer is about 2 m to 4 m thick in Boreholes 1 and 5 and is underlain by a sand and gravel deposit which extends to about Elevation 402.1 m on the south side and Elevation 402.3 m on the north side. Measured 'N' values within the sand and gravel layer ranged from 21 blows to 36 blows per 0.3 m of penetration. The grain size distribution of one sample of the sand and gravel deposit indicates about 60% gravel, 35% sand and 5% silt sizes as shown on Figure 2.

The natural water contents of selected samples of the sand were measured at between 9 percent and 20 percent, with an average of about 16 percent.

4.2.4 Silty Clay Till

The sand and gravel in Boreholes 1 and 5 is underlain by a silty clay till deposit on the north side and a clayey silt deposit on the south side of the road. Measured 'N' values ranged from 15 blows to 24 blows per 0.3 m of penetration within these two deposits indicating a very stiff consistency. The boreholes were terminated at 7.3 m depth, corresponding to 0.3 m penetration into the clayey silt deposit and 1.2 m penetration into the silty clay till deposit.

The silty clay till deposit was also encountered underlying the fill and pavement structure in Boreholes 2 and 3. The till deposit was found to be 1.6 m to 3 m thick at these two borehole locations. Measured 'N' values ranged from 16 to 29 blows per 0.3 m indicating a very stiff consistency.

The grain size distribution curve for one sample of the silty clay till is shown on Figure 3. Atterberg limits testing carried out on two samples of the silty clay till gave a liquid limit of 28 and a plasticity index of 12 for both samples indicating the clay is inorganic and of medium plasticity. The natural water content measured on selected samples of the silty clay till ranged from about 15 percent to 21 percent, with an average of about 17 percent, and were generally at or slightly above the plastic limit.

4.2.5 Silt

The silty clay till in Boreholes 2 and 3 is underlain at 3 m depth by a silt deposit, which contains seams of fine sand and seams of clayey silt. Standard Penetration Testing (SPT) carried out within the silt deposit measured "N" values of 15 to 34 blows per 0.3 m of penetration, indicating a compact to dense state of packing. The natural water content measured on selected samples of the silt ranged from about 12 percent to 21 percent, with an average of about 18 percent.

4.3 Groundwater Conditions

Water levels were noted in the open boreholes during and upon completion of the drilling operation; these levels are shown on the attached record of Borehole sheets. A standpipe piezometer was sealed in Borehole 1 to permit the monitoring of the groundwater level at the culvert site. Details of the piezometer installation and water level measurements are shown on the attached Record of Borehole sheets.

The water level measured in the standpipe installed in Borehole 1 on the north side of the road was at about Elevation 408.5 m. The water level in Borehole 5 on the south side of the road was at about Elevation 408.5 m on completion of drilling. Water was noted in open Boreholes 3 and 4 on completion of drilling at Elevation 413.1 m and 408 m, respectively. Borehole 2, with base at Elevation 421 m, was dry on completion of drilling.

The river water level at the time of the investigation was at about Elevation 408 m, which is consistent with the level, measured in the standpipe piezometer. These measurements indicate that the groundwater table slopes downward toward the north from about Elevation 413 m to about Elevation 408 m. It should be noted that groundwater levels are expected to fluctuate seasonally as well as with the river water level and are expected to be higher during wet periods of the year.

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5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides our recommendations on the geotechnical aspects of design of the proposed twin culvert extension at the Credit River crossing of Broadway Avenue and the overhead sign foundations based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

5.2 Culvert Foundations

It is understood that the proposed works for the culvert carrying Broadway Avenue over Credit River involve extension of the existing twin culvert on the south side only and that the extension will be about 3 m. Based on the General Arrangement drawing for the structure (Drawing No. D-6525-1), the abutments and pier are supported on strip footings founded at about Elevation 405.9 m. The subsoils encountered at this elevation in the borehole put down on the south side of the road consist of the fine to coarse sand deposit. The founding level is at about 2 m below the river water level as measured at the time of the investigation. The river water level as shown on the General Arrangement drawing is at about Elevation 407.5 m.

The sand deposit on which the existing footings are founded is considered extremely sensitive to disturbance due to upward water seepage. Excavation for the footing extension construction without control of the groundwater will result in loss of support to the existing footings as well as loosening of the founding soils for the extension.

5.2.1 Shallow Foundations – Geotechnical Resistance

Assuming that the footing extension is placed on undisturbed sands at Elevation 405.9 m, a factored bearing pressure at ULS of 295 kPa may be assumed for design for a footing width of 1.2 m. The corresponding bearing pressure at SLS may be taken as 175 kPa for 25 mm of settlement.

These values are for vertical concentric loads only. Effects of load inclination and eccentricity need to be taken into account, as appropriate, in accordance with OHBDC utilizing the curve for cohesionless soils.

It should be noted that there will be differential settlement between the existing and proposed extension footing. Given the founding soils, the majority of this settlement is expected to occur immediately after the full load has been applied. It is understood that the proposed extension will be in effect tied to the existing structure by a slab placed on top of the culvert. Some of the settlement under the extension will have occurred by the time that the slab is placed. This will help to distribute loading over the entire length of the abutment and pier footings to accommodate the last portion of differential settlement.

It should be noted that there will be differential settlement between the existing and proposed extension footing. Given the founding soils, the majority of this settlement is expected to occur immediately after the full load has been applied. It is understood that the proposed extension will be in effect tied to the existing structure by a slab placed on top of the culvert. Some of the settlement under the extension will have occurred by the time that the slab is placed. This will help to distribute loading over the entire length of the abutment and pier footings to accommodate the last portion of differential settlement.

5.2.1.1 Resistance to Lateral Forces

Resistance to lateral forces / sliding resistance between the concrete footings and subsoil should be calculated in accordance with Section 6-8.4.3 of the OHBDC assuming an unfactored coefficient of friction between the concrete and the founding soils equal to 0.4.

5.2.1.2 Frost Protection

All footings should be provided with a minimum of 1.2 m of earth cover for frost protection purposes.

5.2.2 Deep Foundations

Considering that the existing culvert is supported on spread footings and that the sands below founding level are susceptible to settlement during driving of piles, the use of piled foundations for the extension is not recommended. In addition, although the spread footing option requires dewatering / groundwater control, the same amount of groundwater control would be required for pile cap construction

5.3 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill and on the subsequent lateral movement of the structure. The following recommendations are made concerning the design of the abutments and the retaining walls in accordance with OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular A or Granular B but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 percent of the material's Standard Proctor maximum dry density.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.
- The granular fill should be placed within the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the footing.
- If the wall support allows lateral yielding of the stem (unrestrained structure), active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (restrained structure), at-rest pressures should be assumed for geotechnical design.
- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- For fill placed within a 60 degree wedge, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular A	Granular B - Type II
Soil Unit Weight	22 kN/m^3	21 kN/m^3
Coefficients of Lateral Earth Pressure		
'active'	0.27	0.31
'at rest'	0.43	0.47

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD-3501.00.

5.4 Construction Considerations – Culvert Footings

In order to construct the footing extension without disturbance of the founding soils, a combination of cofferdam installation and groundwater lowering would probably be required. Sheetpiling for cofferdam construction would have to be driven to below Elevation 402 m in order to obtain a cut-off within the till / silt deposit below the sand and gravel. Unless a complete cut-off and surround can be achieved with a sheetpile cofferdam, groundwater lowering will also be needed to lower the groundwater level to below the base of the excavation. Groundwater lowering can be achieved by pumping from wells screened through the sand and gravel deposit encountered between Elevations 402 m and 404 m.

The groundwater level within the proposed footing extension limits should be lowered to at least 0.5 m below the founding level but no more than 0.8 m below the founding level. This restriction is to ensure that additional settlement of the existing culvert footings does not occur due to consolidation of the sands as a result of the groundwater lowering.

A mud coat should be placed on the founding soils as soon as the founding level had been reached, the base cleaned and the excavation inspected by qualified geotechnical personnel. Inspection of the soils at the founding level should be carried out to confirm that the founding soils are not disturbed, that the base is dry and that the founding stratum is consistent with the conditions assumed for design.

5.5 Overhead Sign Foundations

For design of the overhead sign foundations, the following conditions and parameters may be assumed:

Borehole 2: Station 28+660 Elev. 425 m to base ……	• Compact sandy silt Till underlain by dense silt • Groundwater level below Elev. 421 m	c′ (kPa) 0	φ' 35°	γ (kN/m3) 21
Borehole 3: Station 10+145 Elev. 412.5 m to 411 m …… Elev. 411 m to base ……	 • Very stiff clayey silt Till • Compact silt • Groundwater level at Elev. 413 m	c′ (kPa) 10 0	φ' 34 33°	γ (kN/m3) 21.5 21
Borehole 4: Station 10+370 Elev. 409 m to 407 m …… Elev. 407 m to base …….	 • Compact sand and gravel Fill • Compact sand • Groundwater level at Elev. 408 m	c′ (kPa) 0 0	φ' 32° 34°	γ (kN/m3) 21.5 21

At the location of Boreholes 3 and 4, excavating for caisson construction for the sign foundations will be extended into silts and sands below the groundwater level. The excavations should be carried out within temporary liners placed tight to the sides to maintain the soils in place without disturbance. Loosening of the soils at the base of the excavation will occur unless lowering of the groundwater level is carried out prior to excavation. Groundwater lowering is considered feasible by pumping from wells installed adjacent to the footing excavations at Borehole 4 where fine to coarse sands were encountered. Lowering of the groundwater level at the location of Borehole 3 would likely only be possible with the use of eductors. Consideration could be given to minimizing the disturbance of the soils at the base of the caissons by maintaining the head of water in the excavation at all times excavation and concrete placement such that upward flow does not occur at the base. Placement of the concrete using tremie techniques would be required.

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WORD S/FINALDAT/1100/991-1171/2000/991171AR1

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.).

Dynamic Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, s_u kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_S)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

Note:
1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π = 3.1416
ln x, natural logarithm of x
$\log_{10}$ x or log x, logarithm of x to base 10
g acceleration due to gravity
t time
F factor of safety
V volume
W weight

II. STRESS AND STRAIN

γ shear strain
Δ change in, e.g. in stress: $\Delta\sigma$
ε linear strain
ε_v volumetric strain
η coefficient of viscosity
ν Poisson's ratio
σ total stress
σ' effective stress ($\sigma' = \sigma - u$)
σ'_{vo} initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$ principal stresses (major, intermediate, minor)
σ_{oct} mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ shear stress
u porewater pressure
E modulus of deformation
G shear modulus of deformation
K bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$ bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$ dry density (dry unit weight)
$\rho_w(\gamma_w)$ density (unit weight) of water
$\rho_s(\gamma_s)$ density (unit weight) of solid particles
γ' unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R relative density (specific gravity)of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e void ratio
n porosity
S degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

w water content
w_l liquid limit
w_p plastic limit
I_P plasticity Index $= (w_l - w_p)$
w_S shrinkage limit
I_L liquidity index $= (w - w_p) / I_p$
I_C consistency index $= (w_l - w) / I_p$
e_{max} void ratio in loosest state
e_{min} void ratio in densest state
I_D density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(c) Hydraulic Properties

h hydraulic head or potential
q rate of flow
v velocity of flow
i hydraulic gradient
k hydraulic conductivity (coefficient of permeability)
j seepage force per unit volume

(d) Consolidation (one-dimensional)

C_c compression index (normally consolidated range)
C_r recompression index (overconsolidated range)
C_S swelling index
C_α coefficient of secondary consolidation
m_v coefficient of volume change
c_v coefficient of consolidation
T_v time factor (vertical direction)
U degree of consolidation
σ'_p pre-consolidation pressure
OCR Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

(e) Shear Strength

τ_p, τ_r peak and residual shear strength
ϕ' effective angle of internal friction
δ angle of interface friction
μ coefficient of friction $= \tan\delta$
c' effective cohesion
c_u, S_u undrained shear strength ($\phi = 0$ analysis)
p mean total stress $(\sigma_1 + \sigma_3)/2$
p' mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q $(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u compressive strength $(\sigma_1 - \sigma_3)$
S_t sensitivity

Notes: 1. $\tau = c' + \sigma' \tan\phi'$
2. Shear strength = (Compressive strength)/2

RECORD OF BOREHOLE No 1

METRIC

PROJECT 991-1171

W.P. 449-97-00 | LOCATION STA. 9+885m ; 19.0m LT. CL, HWY 9 | ORIGINATED BY DJM

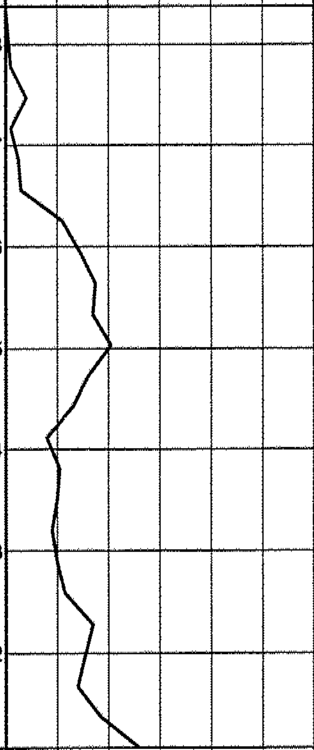
DIST 3 HWY 10 | BOREHOLE TYPE CONTINUOUS FLIGHT SOLID AND HOLLOW STEM AUGERS & CONE TEST | COMPILED BY BG

DATUM GEODETIC | DATE 11.4.99 - 11.4.99 | CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
408.38	GROUND SURFACE						
0.00	Peat, fine fibrous Soft Black				∇ (408)		
		1	SS	4			
406.86							
406.70 1.68	Organic Silt Soft Grey	2	SS	9			
406.25 2.13	Silty fine Sand Compact Grey						
	Sand, fine to coarse, trace silt, some gravel Loose to compact Grey	3	SS	14			
		4	SS	11		o	13 86 1
		5	SS	5			
403.96 4.42	Sand, fine, some silt to silty sand, trace gravel Compact Grey	6	SS	12		o	
402.74 5.64	Sand and Gravel, trace silt, occ. cobbles Compact Grey	7	SS	21			
402.28 6.10	Silty Clay, some sand, trace gravel Stiff to very stiff Brown/Grey (Till)	8	SS	15		Wp–w–WL	
		9	SS	19			
401.06 7.32	END OF BOREHOLE						

Water level in standpipe at Elev.407.9m on Nov.4,1999 and at Elev.408m on Nov.5,1999.

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100); ELEVATION SCALE 408–402; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³



+³, ×³: Numbers refer to Sensitivity   O ³% STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 2

METRIC

PROJECT 991-1171

W.P. 449-97-00

LOCATION STA. 28+659m ; 2.5m RT. CL, HWY 10

ORIGINATED BY DJM

DIST 3 HWY 10

BOREHOLE TYPE CONTINUOUS FLIGHT SOLID STEM AUGER

COMPILED BY BG

DATUM GEODETIC

DATE 11.4.99 - 11.4.99

CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR / SA / SI / CL
426.12	PAVEMENT SURFACE					
425.82 / 0.00	Asphalt					
425.66	Granular A Roadbase					
425.42	Granular B Subbase					
0.70	Silty Clay, trace to some sand, Very stiff, Brown (Till)	1	SS	17		
		2	SS	19	o	- / 5 / 82 / 13
		3	SS	25		
422.46		4	SS	29	o	
3.66	Silt, trace sand, with clayey silt seams, Dense, Brown	5	SS	33		
421.09		6	SS	34	o	
5.03	END OF BOREHOLE					
	Borehole dry during drilling Nov. 4, 1999					

Chart headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (426, 425, 424, 423, 422); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity

$O^{3\%}$ STRAIN AT FAILURE

ON_MOT 991-1171.GPJ ON_MOT.GDT 28/1/00

RECORD OF BOREHOLE No 3

METRIC

PROJECT 991-1171			
W.P. 449-97-00	LOCATION STA. 10+146m ; 6.0m LT. CL. HWY 10		ORIGINATED BY DJM
DIST 3	HWY 10	BOREHOLE TYPE CONTINUOUS FLIGHT SOLID STEM AUGER	COMPILED BY BG
DATUM GEODETIC	DATE 11.4.99 - 11.4.99		CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
413.99	GROUND SURFACE				
0.00	Sand and gravel, some silt, with cobbles Dense Brown (Fill)	1	SS	44	∇
412.62 / 1.37	Silty Clay, trace to some sand, trace gravel Very stiff Brown (Till)	2	SS	26	
		3	SS	16	
410.94 / 3.05	Silt, trace sand and clay, with fine sand seams Compact Brown	4	SS	15	
410.33 / 3.66	Silt, trace sand Compact Brown	5	SS	19	
		6	SS	26	
408.96 / 5.03	END OF BOREHOLE				

Water level in open borehole at Elev. 413.1m during drilling Nov. 4, 1999

Elevation scale: 413, 412, 411, 410, 409

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED: 20, 40, 60, 80, 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10, 20, 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 4

METRIC

PROJECT 991-1171
W.P. 449-97-00
LOCATION STA. 10+370m ; 1.0m RT. CL, HWY 10
ORIGINATED BY DJM
DIST 3 HWY 10
BOREHOLE TYPE CONTINUOUS FLIGHT SOLID STEM AUGER
COMPILED BY BG
DATUM GEODETIC
DATE 11.5.99 - 11.5.99
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
410.86	PAVEMENT SURFACE						
410.80 0.06	Asphalt						
410.46	Granular A Roadbase						
0.40	Sand and gravel, trace silt, trace cobble with pockets of clayey silt Compact to dense Brown (Fill)		1	SS	49		410
409.03			2	SS	11		409
408.73	Silt, trace sand, trace gravel (Fill)						
2.13	Sand and gravel, trace silt Loose to dense Brown (Fill)		3	SS	37		
						∇	408
407.20			4	SS	6		
3.66	Sand, fine to coarse, trace silt some gravel Compact Grey		5	SS	19		407
405.83			6	SS	21		406
5.03	END OF BOREHOLE						

Water level in open borehole at Elev.408m during drilling Nov. 5, 1999

Other column headings (no data plotted): DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa — O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 4A

METRIC

PROJECT 991-1171			
W.P. 449-97-00	LOCATION STA. 10+450m ; 1.5m RT. CL, HWY 10		ORIGINATED BY DJM
DIST 3	HWY 10	BOREHOLE TYPE CONTINUOUS FLIGHT SOLID STEM AUGER	COMPILED BY BG
DATUM GEODETIC	DATE 11.5.99 - 11.5.99		CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION (PAVEMENT SURFACE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.00	Asphalt										
-0.15 / 0.15	Granular A Roadbase										
0.37	Sand and gravel, trace silt, trace cobble Compact Brown (Fill)										
-1.68 / 1.68	Clayey silt, trace sand some gravel Stiff Brown (Fill)		1	SS	14						
-2.13 / 2.13	Sand and gravel, trace silt with silt pockets Compact Brown (Fill)		2	SS	15	∇					
			3	SS	19						
-5.03 / 5.03	END OF BOREHOLE Water level in open borehole at 2.90m depth during drilling Nov. 5, 1999										

Legend: SHEAR STRENGTH kPa — O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × REMOULDED. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L.

+³, ×³: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 5

METRIC

PROJECT 991-1171

W.P. 449-97-00 | LOCATION STA. 9+908m ; 11.5m RT. CL, HWY 9 | ORIGINATED BY DJM

DIST 3 HWY 10 | BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER | COMPILED BY BG

DATUM GEODETIC | DATE 11.5.99 - 11.5.99 | CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
409.10	GROUND SURFACE					409		
0.00	Sand and gravel, trace cobble, Brown (Fill)							
408.49 / 0.61	Silt, trace organics, rootlets, peat, Very loose, Brown (Fill)	1	SS	4	∇	408		
408.03 / 1.07	Sand and gravel, trace silt and gravel, Loose, Grey (Fill)	2	SS	10		407		
		3	SS	10				
406.20 / 2.90	Sand, fine to medium, some silt to silty sand, trace gravel, Compact, Grey	4	SS	12		406		
		5	SS	17		405	~20	- 65 33 2
		6	SS	15				
403.92 / 5.18	Sand and Gravel, trace silt, with occ. cobbles, Dense, Grey	7	SS	31		404		
		8	SS	36		403	~10	58 35 7
402.09 / 7.01	Clayey Silt, trace sand with sand seams, Very stiff, Grey	9	SS	24		402		
401.78 / 7.32	END OF BOREHOLE							

Water level in open borehole at Elev. 408.49m during drilling Nov. 5, 1999

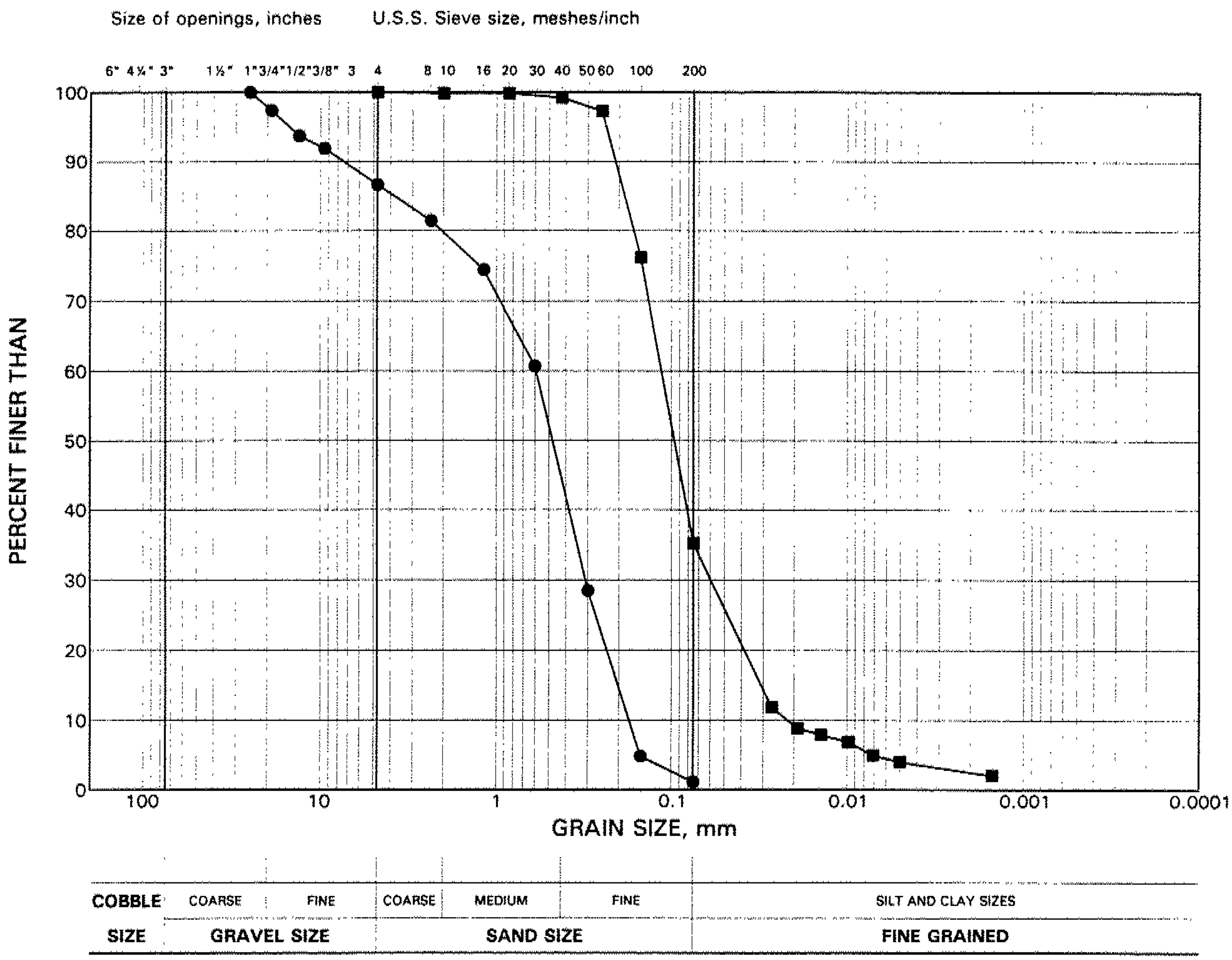
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

OVERSIZE DRAWING(S)

GRAIN SIZE DISTRIBUTION
Sand, some gravel to Silty Sand

FIGURE 1



LEGEND

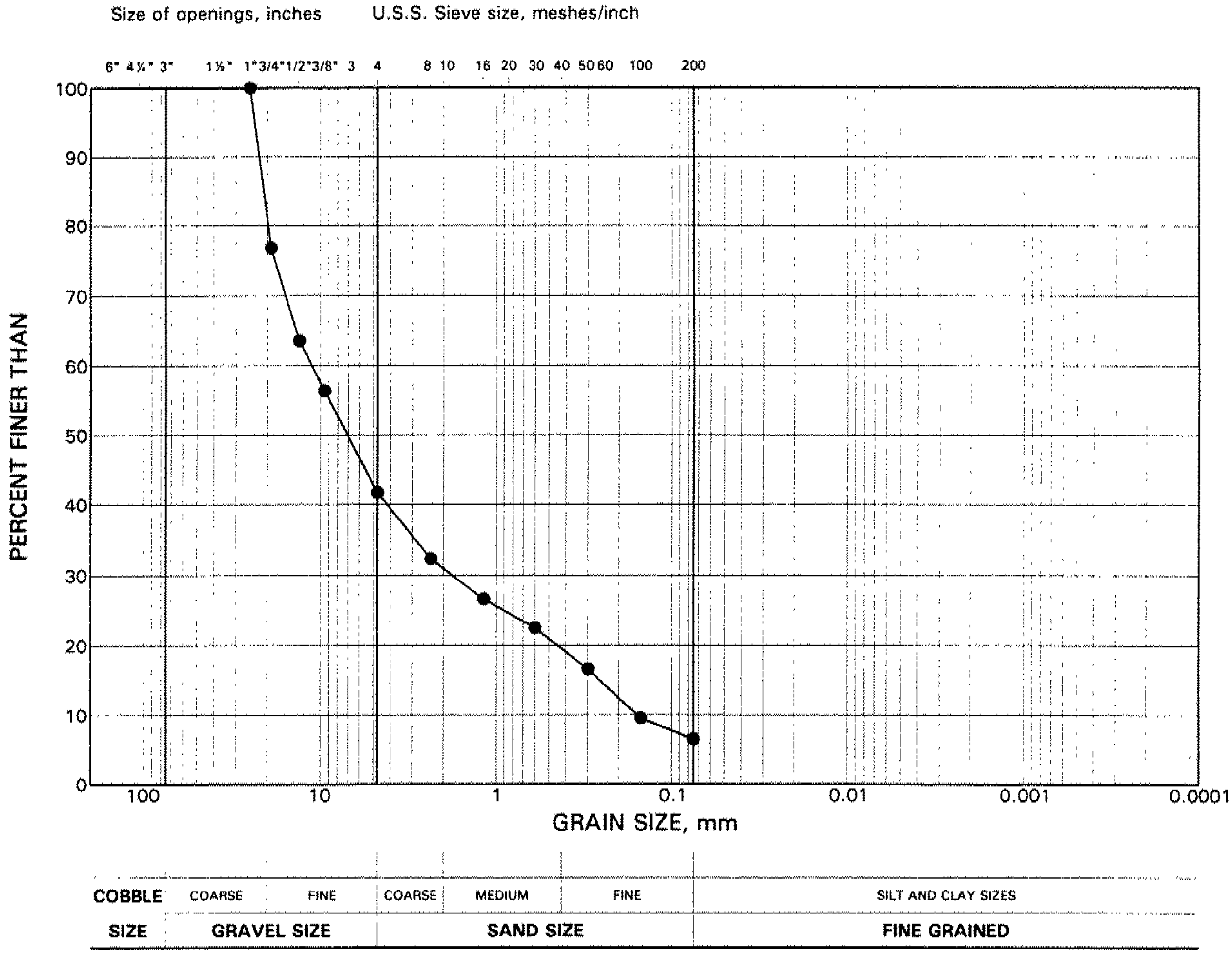
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	1	4	405.1
■	5	5	405.1

Project 991-1171

Golder Associates

GRAIN SIZE DISTRIBUTION
Sand and Gravel

FIGURE 2

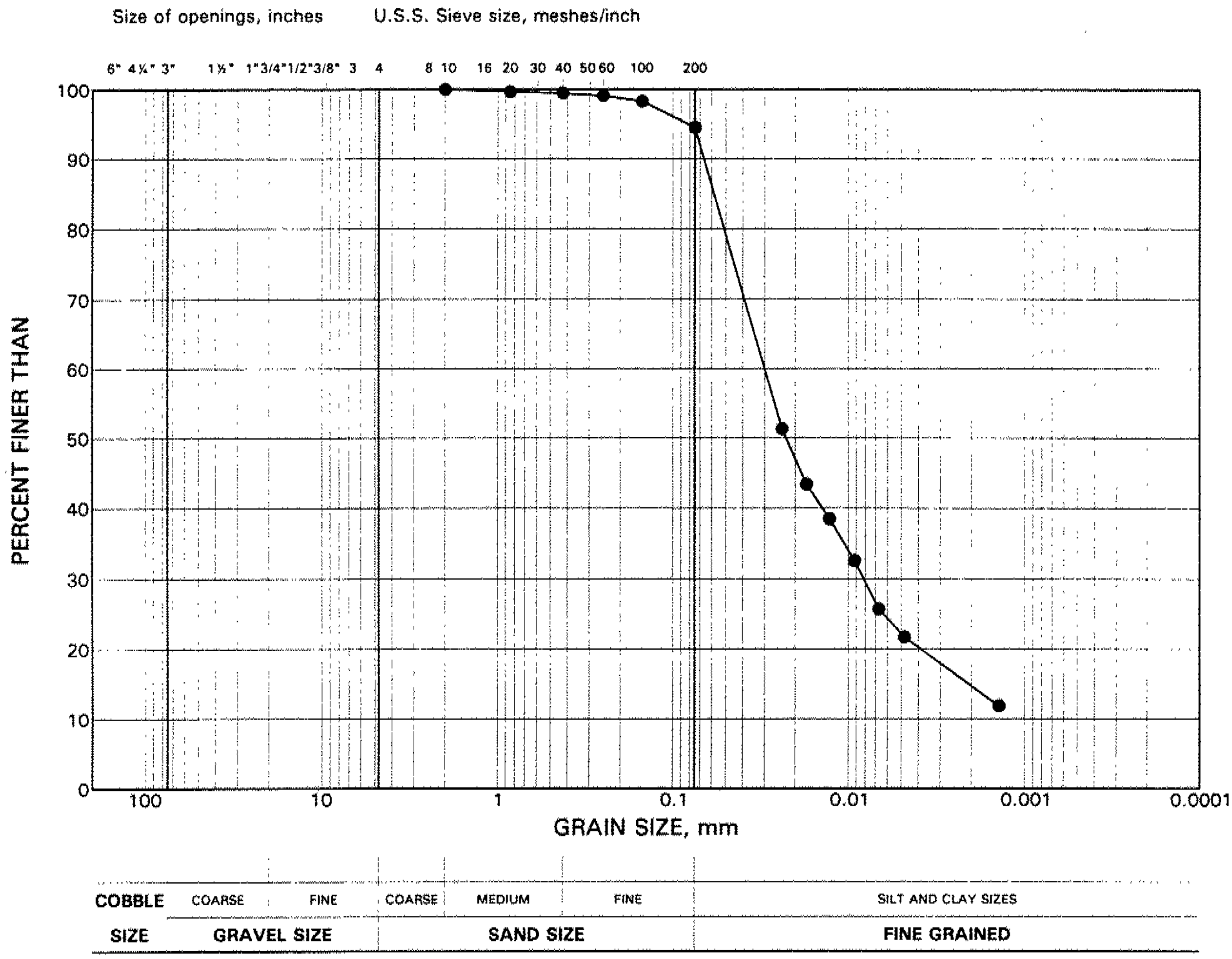


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	5	8	402.8

GRAIN SIZE DISTRIBUTION
Silty Clay Till

FIGURE 3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	2	2	424.0

Project 991-1171

Golder Associates

GEOCRES No 40P16-1

#56-F-204C

HWY # 10

CREDIT RIVER

BRIDGE

BA 553

RACEY, MacCALLUM AND ASSOCIATES
LIMITED

A COMPANY OWNED, DIRECTED AND OPERATED BY

Consulting Engineers
AND ASSOCIATED STAFF

MONTREAL VANCOUVER
TORONTO

20 CARLTON STREET

Reference: S-5.-628/1-1

Toronto, Ontario,
November 13, 1956.

A. A. Toye, Esq.,
Bridge Engineer,
Department of Highways of Ontario.
280 Davenport Road,
Toronto, Ontario.

Attention Mr. S. McCombie

Dear Sirs:

Further to your recent request, we have pleasure in enclosing herewith two additional copies of our report on the foundation investigation for the proposed Credit River Bridge, Station 555+70, Highway 10, Ontario. This investigation was performed at the request of A. M. Dickinson & Associates.

We trust that these will meet your requirements.

Yours very truly,

RACEY, MacCALLUM AND ASSOCIATES LIMITED

H. A. Trow, P. Eng.,
Divisional Soils Engineer

HD/KA
Enclosures

BA-553

C
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P
Y

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AND ASSOCIATED STAFF

MONTREAL VANCOUVER
TORONTO

DONALD C. MacCALLUM, B.ENG., M.E.I.C., P.ENG
H. JOHN RACEY, B.SC., M.E.I.C., P.ENG
A. ERIC SANKINE, B.SC., M.E.I.C., A.M.I.ELEC.E., P.ENG

TORONTO DIVISION
20 CARLTON STREET

26 October 1956.

Reference: S-500-629/T-488

Department of Highways of Ontario,
A.W. Dickinson and Associates,
38 Irwin Avenue,
TORONTO, Ontario.

Attention: Dr. A. W. Dickinson

FOUNDATION INVESTIGATION FOR THE
PROPOSED CREDIT RIVER BRIDGE,
STATION 555+70, HIGHWAY 10, ONTARIO

Dear Sirs:

We have completed our investigation of the subsoil conditions at the above noted site and our report on the subject is attached hereto. For your convenience, the conclusions of this report are repeated, as follows:-

1. The subsoil at the present bridge site consists of a surface layer of compressible peaty topsoil, underlain by saturated silt. The desirable load bearing level is at, or below El. 1311 feet, where a stratum of dense gravel, sand, and small boulders was encountered.

2. The recommended safe bearing value for this coarse granular stratum is of the order of 5000 p.s.f., for a limiting settlement of one inch.

3. Piles should encounter refusal in this granular material, although the exact depth at which this occurs may vary somewhat, due to the random nature of the small boulders. Ultimate refusal depth however, should not extend below El. 1288 feet.

4. Excavation difficulties below river level, in the upper silt layer, should be anticipated.

We thank you for this opportunity to be of service to you, and shall be pleased to discuss any matters not specifically covered in this report, at your convenience.

Yours very truly,
RACEY, MacCALLUM AND ASSOCIATES LIMITED

W. A. Trow, P. Eng.
Divisional Soils Engineer

FOUNDATION INVESTIGATION FOR THE PROPOSED CREDIT RIVER BRIDGE, STATION 555+70, HIGHWAY 10, ONTARIO.

Reference: S-500-629/T-488

Racey, MacCallum and Associates Limited

26 October, 1956.

RACEY, MacCALLUM AND ASSOCIATES LIMITED

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DESCRIPTION OF THE SITE AND SUBSOIL CONDITIONS	1
DISCUSSION OF THE RESULTS	2
CONCLUSIONS	2

ENCLOSURES

SKETCH OF THE SITE, SHOWING LOCATION OF THE BOREHOLES	1
ENGINEERING DATA SHEETS	2 & 3

26 October 1956.

FOUNDATION INVESTIGATION FOR
THE PROPOSED CREDIT RIVER
BRIDGE, STATION 555+70,
HIGHWAY 10, ONTARIO

This report contains the results of a foundation investigation consisting of two borings, carried to a depth of thirty-five to forty feet, near the abutments of the present bridge at the above noted site. This work was carried out during the period from 17 October to 23 October 1956. The location of the borings is indicated on enclosure No. 1.

DESCRIPTION OF THE SITE AND SUBSOIL CONDITIONS

The site of the proposed bridge and of the existing highway bridge lies in a relatively flat valley, approximately one mile wide. The valley is bounded to the north and south by steep hills which appear to consist of dense glacial till deposits, with random limestone boulders at the surface. Conversation with a local well-driller indicates that limestone bedrock was encountered at a depth of about fifty feet in these hills, although much greater bedrock depths were noted near Mournahan's Snack Bar, just south east of Orangeville. In the vicinity of the river itself, a wide flat alluvial plain has been developed, suggesting that the river has progressed across it in geologically recent times. The present bridge shows evidence of concrete deterioration, and occasional chunks of concrete were noted to fall into the river upon the passage of heavy trucks.

The subsoil at the site is shown in the profiles for borings 1 and 2, enclosures 2 and 3. Briefly, the profile consists of very peaty topsoil to El. 1321.5, underlain by approximately nine feet of loose, slightly cohesive silt which, in turn, changes at about El. 1311 feet to dense fine to coarse gravel with some boulders, which continued to El. 1280 feet, the ultimate depth of the borings. This granular material was very difficult to penetrate and sampling in it was very unsatisfactory. In hole No. 1 this material was broken near El. 1304 feet, by approximately seven feet of dense silt with some gravel. The water table at both locations coincided with river level.

The relative density of the various strata was determined by penetration measurements, using the standard split spoon, the two inch diameter cone and by recording the blows per foot required to drive the three inch casing. Undisturbed shelby tube samples of the silt were also taken, for determinations of relative density. The relative density of the upper silt was found to be of the order of 77 percent, which indicates a dense state of compaction and conflicts with the standard penetration measurements. This paradox has been noted on other occasions and gives evidence of the conservativeness of the empirical standard penetration test under certain circumstances.

DISCUSSION OF THE RESULTS

In view of the subsoil conditions observed at the site, the alternatives for the support of the proposed highway bridge would appear to be somewhat limited. Because of the flood conditions on record for this part of the river, and the probable low resistance to scour of the upper stratum of silt, it would appear desirable to carry any foundation down to the gravel deposits encountered in the vicinity of El. 1311 feet. This could be done either by excavating to this depth and installing the required abutment footings, or by driving short piles to refusal in the gravel stratum. The former method of construction would involve the usual difficulties associated with excavating below the water table in silt, and well-shored sheet piling, driven into the gravel, would be required to contain this unstable material. A free flow of water from the underlying gravel should also be anticipated.

According to the standard penetration measurements in the gravel, sand and boulders, the empirical safe bearing value for an abutment footing founded at El. 1311 feet is of the order of 5000 p.s.f. This value allows for the effect of submergence of the gravel, and also for the fact that large abutment footings, because of their rigid nature, undergo less settlement than ordinary footings for similar soil conditions. It does not allow for the fact that granular deposits adjust instantaneously to any load application and, therefore, that a considerable amount of the total one inch settlement will take place as the abutment is being constructed.

If driven to refusal, the ultimate bearing capacity of piles will be governed by structural considerations. A factor of safety of three should be applied to this ultimate capacity. As a check on this capacity, current soil mechanics literature indicates an ultimate capacity for the granular material with the penetration resistances noted at this site, of the order of one hundred times the tip area of the pile. Although wood piles would appear to be an economic method of abutment support, some danger of pile brooming on some of the small boulders should be anticipated.

CONCLUSIONS

On the basis of the foregoing comments, the following conclusions regarding subsoil conditions, can be drawn.

1. The subsoil at the present bridge site consists of a surface layer of compressible peaty topsoil, underlain by saturated silt. The desirable load bearing level is at, or below, El. 1311 feet, where a stratum of dense gravel, sand, and small boulders was encountered.

2. The recommended safe bearing value for this coarse granular stratum is of the order of 5000 p.s.f., for a limiting settlement of one inch.

3. Piles should encounter refusal in this granular material, although the exact depth at which this occurs may vary somewhat, due to the random nature of the small boulders. Ultimate refusal depth however, should not extend below El. 1238 feet.

4. Excavation difficulties below river level, in the upper silt layer, should be anticipated.

W. A. Trow, P. Eng.,
Divisional Soils Engineer.

WAT/MD
Copy /PK

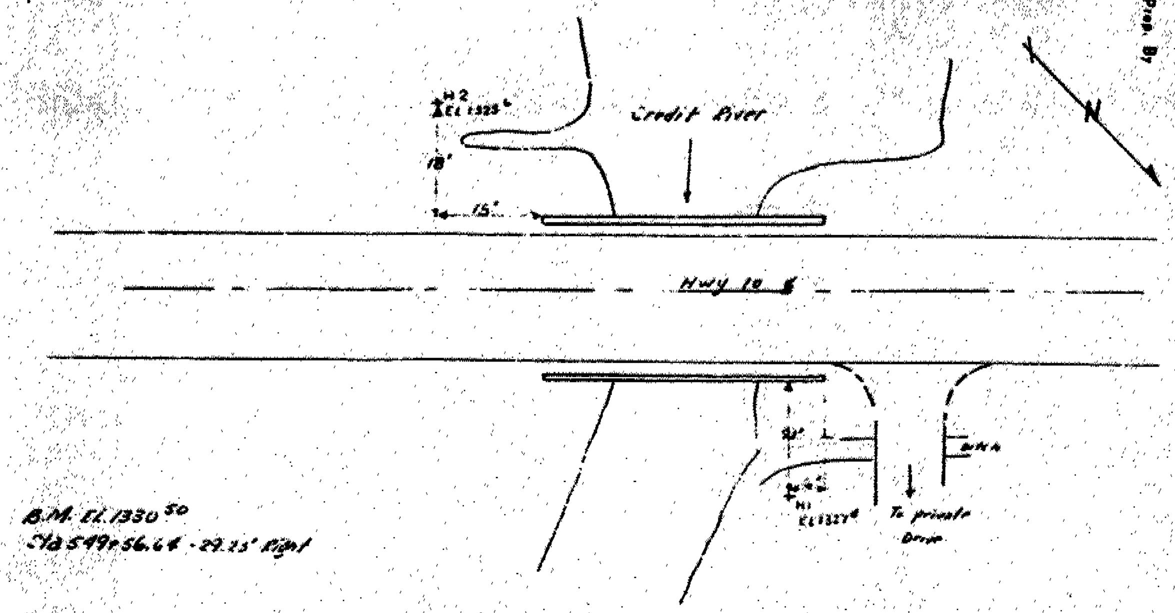
Original and 3 copies - Department of Highways of Ontario,
A. W. Dickinson & Associates, Toronto.

Racey, MacCallum & Associates Ltd.

Racey, MacCallum & Associates Ltd.

Order No.

Prep. By



Sketch of site Showing Location of Boreholes

Enclosure No. 1

Encl. No. 2

Order No. 5520-629/7408 **TRACEY, MacCALLUM AND ASSOCIATES** LIMITED

Foundation Engineering Division

Hole Begun ______   Hole Ended ______

Engineering Data Sheet for Borehole: ______

Driller ______   Helper ______   Checked by ______

Job Name: Credit River Bridge Hwy No 10 Sta 555+70

Job Located: Approx. 2 mi South Orangeville Ont

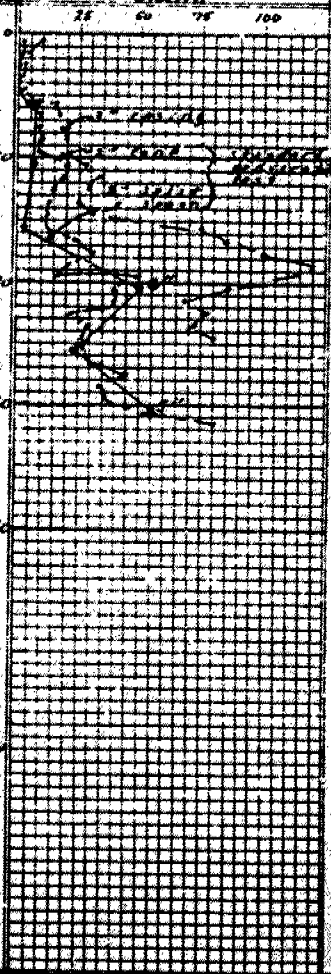
Hole Located: See Encl. No 1

Hole Elevation: 1327.4 Datum: Geodetic

Day ______ Month ______ Year ______

Depth	El.	Thickness	Symbol	Description	Sampling Method
	1327.4			Ground surface	
6'				Black peaty topsoil	
				Uniform loose grey silt (slightly cohesive)	SS.1
					SS.2
16'	1311			Fine to coarse gravel	SS.3
23'				Uniform dense grey silt with gravel	SS.4
					SS.6
40'				Coarse gravel & small stones (wash water clear). Drilled with AX bit through small boulders & gravel to 40'	SS.6
				End of bore	

Tabular Values: Blows/Ft. — 25, 50, 75, 100



Note: water level coincides with river level

Legend

SS = 2" split spoon

Tube = 2" Shelby tube

Blows/ft. 140# hammer 30" drop

Order No. 5520-629/7408

Enclosure No. 1

Encl No 9

Order No.: [illegible]-621/1960 **RACEY, MacCALLUM AND ASSOCIATES**
LIMITED
Foundation Engineering Division

Hole Begun ________

Hole Ended ________ **Engineering Data Sheet for Borehole:** 2

1 B.
Driller

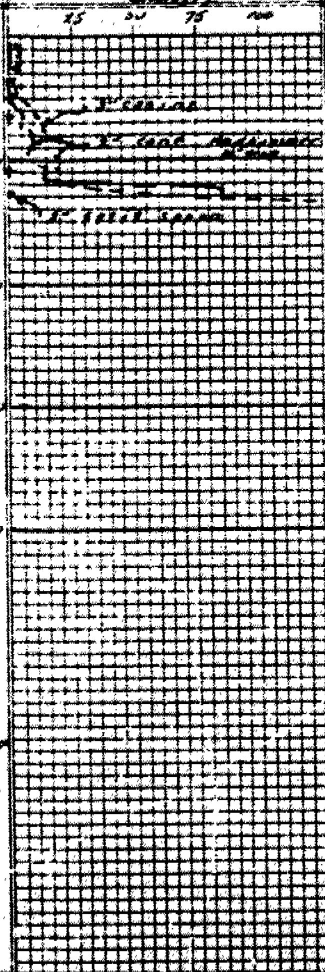
Helper

Checked by

Job Name: Credit River Bridge Hwy No 10 Sta 555 + 70
Job Located: Approx 2 mi South Orangeville Ont
Hole Located: See Encl. No 1
Hole Elevation: 1325.6 Datum: Geodetic

Day Month Year

DEPTH	EL.	THICKNESS	SYMBOL	DESCRIPTION	PENETRATION VALUES Blows/ft	SAMPLING METHOD
0	1325.6			Ground Surface	0	
				Black topsoil		
≈4				Loose medium sand		
6						Tw 1 / SS 2
				Loose slightly cohesive silt	10	Tw 3
13.5	1312					SS 4. 135 157
				Dense gravel & boulders and coarse sand (Sampling impossible)		180 156 120 150 170
					20	145
				Some silt in evidence at 23 ft.		200 205 75
						100 225 205 205 250 350
31					30	
				Drill with NXT bit through dense gravel & boulders to 35'.		Casing Blows
36'	1289			End of Bore		
					40	



Note: water table coincides with river level

Legend
SS = 2" split spoon
Tw = 2" shelby tube
Blows/ft. 140# hammer 30" drop

GEOCRES 40P16-2

#56-F-205C

Hwy #10

CULVERT AT

CREDIT RIVER

B.A.549

RACEY, MacCALLUM AND ASSOCIATES
LIMITED

A COMPANY OWNED, DIRECTED AND OPERATED BY

Consulting Engineers
AND ASSOCIATED STAFF

MONTREAL — VANCOUVER
TORONTO

DONALD C. MacCALLUM, B.ENG., M.E.I.C., P.ENG.
H. JOHN RACEY, B.SC., M.E.I.C., P.ENG.
A. ERIC RANKINE, B.SC., M.E.I.C., A.M.I.ELEC.E., P.ENG.

TORONTO DIVISION
20 CARLTON STREET

Reference: S-500/T-487

5 November 1956.

Mr.A.M.Toye,
Bridge Engineer,
Department of Highways of Ontario,
280 Davenport Avenue,
TORONTO, Ontario.

Attention: Mr.S.McCombie

RE: FOUNDATION INVESTIGATION FOR A CULVERT, AT THE CREDIT RIVER, HIGHWAY 10 CROSSING, APPROXIMATELY FOUR MILES NORTH OF CALEDON,ONTARIO.

Dear Sirs:

We have completed our investigation of subsoil conditions at the above noted site, and our report on the subject is attached hereto.

For your convenience, the conclusions of this report are as follows:-

1. Neglecting the organic materials above El.1311.5 feet, the subsoil at the site consists of uniform coarse grey silt, which is separated between El.1299.5 and 1292.5 feet by a layer of coarse gravel and boulders. The silt appears to be in a much denser condition below El.1292 feet, although there is evidence to suggest that the relative density of the upper silt is greater than standard penetration tests indicate.

2. The permissible bearing value and recommended footing depth, depends upon the type of culvert under consideration. A reinforced concrete box culvert would appear to require support on the dense gravel and boulder stratum encountered below El.1299 feet. The bearing capacity of this stratum and of the underlying silt, is quite high for either a rectangular footing or for short piles. Excavation difficulties in the silt below the river level, should be anticipated.

5 November 1956.

3. A corrugated steel culvert, founded on the upper silt, should be possible, provided a bed of well-graded sand and gravel is placed in the river before the water level is lowered. Filter and rip-rap protection at the entrance and exist of the culvert is also recommended. This protection should be carried above maximum anticipated flood level.

We shall be pleased to discuss any aspect of the foundation conditions at this site, in greater detail, if you so require, or if culvert designs other than assumed here are given consideration. We thank you for this opportunity to be of service to you.

Yours very truly,
RACEY,MacCALLUM AND ASSOCIATES LIMITED

W. Trow

W.A.Trow,P.Eng.,
Division Soils Engineer.

WAT/MD

RACEY, MacCALLUM and ASSOCIATES LIMITED

FOUNDATION INVESTIGATION FOR A CULVERT,

AT THE CREDIT RIVER, HIGHWAY 10 CROSSING,

APPROXIMATELY FOUR MILES NORTH OF

CALEDON, ONTARIO.

Report No: S-500/T-487 Racey,MacCallum and Associates Limited

5 November 1956.

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5 November 1956.

FOUNDATION INVESTIGATION FOR A CULVERT, AT THE CREDIT RIVER, HIGHWAY 10 CROSSING, APPROXIMATELY FOUR MILES NORTH OF CALEDON, ONTARIO.

This report describes the results of a foundation investigation, consisting of two borings taken to a depth of forty two feet below the present ground surface, to determine the subsoil competence at the above noted culvert site. This work was carried out during the period from 10 October to 16 October, 1956, and was restricted to two borings because uniform subsoil conditions appeared to prevail. The locations of these borings, designated H2 and H3, are indicated on the attached sketch, enclosure no.1.

DESCRIPTION OF THE SITE AND OF THE SUBSOIL

The Credit River at this site, flows in a westerly direction along the northern base of a high hill, which rises toward the south and which appears to consist of densely-packed glacial till deposits, with erratic limestone boulders resting on its surface. A local well-driller reported that limestone bedrock lies about fifty feet below the surface of the top of this hill and, therefore, exists at a considerably higher elevation than present river level. In flood periods the river is reported to rise to each bank of its flood plain to the west, and to cover an adjacent access bridge, which is thirty two inches above present river level. The flood rise is, therefore, probably of the order of four feet.

The existing bridge shows no evidence of fatigue due to poor foundation conditions, and the present deterioration of the concrete in the structure is due to other causes. Residents in the area during construction of this bridge, believe that it was founded in the dense gravel which was encountered at El.1299 feet in this investigation.

The subsoil profiles, as determined by the two borings, are indicated in enclosures 2 and 3, and are relatively self-explanatory. Hole 2 showed a surface layer of six and one half feet of very spongy topsoil, which remained dry until the underlying sand and gravel stratum was encountered. The ground water then rose immediately to a level coinciding with river level. This coarse granular stratum had a thickness of approximately four and one half feet and was underlain by uniform grey silt, which extended to El.1299.3 feet. According to the empirical standard penetration test, which determines relative density on the basis of the driving resistance per foot required to drive a standard two inch O.D.split-spoon, under 350 foot pounds energy, this silt may be classed as loose to medium dense. However, measurements of its density, as determined on undisturbed shelby tube samples, indicates that it has a relative density of the order of 77 percent of the maximum possible obtainable. This was also noted for the silt deposits in the bridge investigation made approximately half a mile north of this site.

Below the silt between El.1299.3 and 1292.3 feet, was a seven foot stratum of dense gravel and boulders which, in turn, was underlain by uniform dense grey silt. This latter material continued at least to El.1278 feet, the maximum depth of the boring. The subsoil profile observed in hole 2 was closely duplicated in hole 3, near the north east corner of the bridge. The only difference between the two borings was the absence of the four foot sand and gravel stratum at El.1314 feet, and the less spongy nature of the topsoil in hole 3.

DISCUSSION OF THE RESULTS

Any recommendations regarding the most desirable depth for the support of the proposed culvert and the safe bearing value at this depth, will be determined by the type of culvert under consideration. The foundation problems associated with the support of a reinforced-concrete box culvert for example, will conform in large part, to those requiring attention in the design of a regular bridge structure. In that instance, the most suitable depth for transmitting the footing load is at El.1299 feet, the top of the gravel and boulder stratum. The bearing capacity at this depth will be quite high and of the order of 8000 p.s.f. for a rectangular footing. For short piles it is probable that refusal will be encountered in this coarse granular soil, although some piles may penetrate into the underlying silt. The safe bearing capacity, in tons, of such piles will be approximately equal to thirty times the area in square feet of each pile point. A load test would be required for greater precision.

The silt overlying the gravel and boulder stratum is not recommended as a medium of support, despite earlier opinions regarding its probable in-situ relative density, because of the danger of obtaining a looser or quick condition during excavation below the water table. Control of a quick-sand condition at this site would only be obtainable by a combination of a cofferdam to hold back the river, and a vacuum well-point system to stabilise the silt. Because of the low permeability of the silt, this system would have to be in operation for a few days before excavation could proceed.

Because of its flexible nature, a corrugated steel culvert is more tolerant of the foundation conditions it requires and, therefore, support on the upper stratum of silt would appear to be quite permissible. Installation of this type of culvert in the dry state, must also anticipate a quick sand condition in the silt, unless control measures are taken. One method of control is to place a layer of well-graded sand and gravel in the river bed before the water is diverted. Some dredging may be required to clean the river bed prior to installation of this material, and about two feet of this mixture would be necessary to overcome any segregation of particles and to ensure that a good filter is obtained. This filter should help to control the movement of the silt after the river has been diverted for construction purposes. The use of a well-graded filter and rip-rap at the entrance and exit of the culvert, will also be necessary to avoid the undermining effects of stream flow after the culvert is in operation.

CONCLUSIONS

The comments presented in the foregoing section can be summarised as follows:-

1. Neglecting the organic materials above El.1311.5 feet, the subsoil at the site consists of uniform coarse grey silt, which is separated between El.1299.5 and 1292.5 feet by a layer of coarse gravel and boulders. The silt appears to be in a much denser condition below El.1292 feet, although there is evidence to suggest that the relative density of the upper silt is greater than standard penetration tests indicate.

2. The permissible bearing value and recommended footing depth depends upon the type of culvert under consideration. A reinforced concrete box culvert would appear to require support on the dense gravel and boulder stratum encountered below El.1299 feet. The bearing capacity of this stratum and of the underlying silt, is quite high for either a rectangular footing or for short piles. Excavation difficulties in the silt below the river level, should be anticipated.

3. A corrugated steel culvert, founded on the upper silt, should be possible, provided a bed of well-graded sand and gravel is placed in the river before the water level is lowered. Filter and rip-rap protection at the entrance and exit of the culvert is also recommended. This protection should be carried above maximum anticipated flood level.

W. Trow

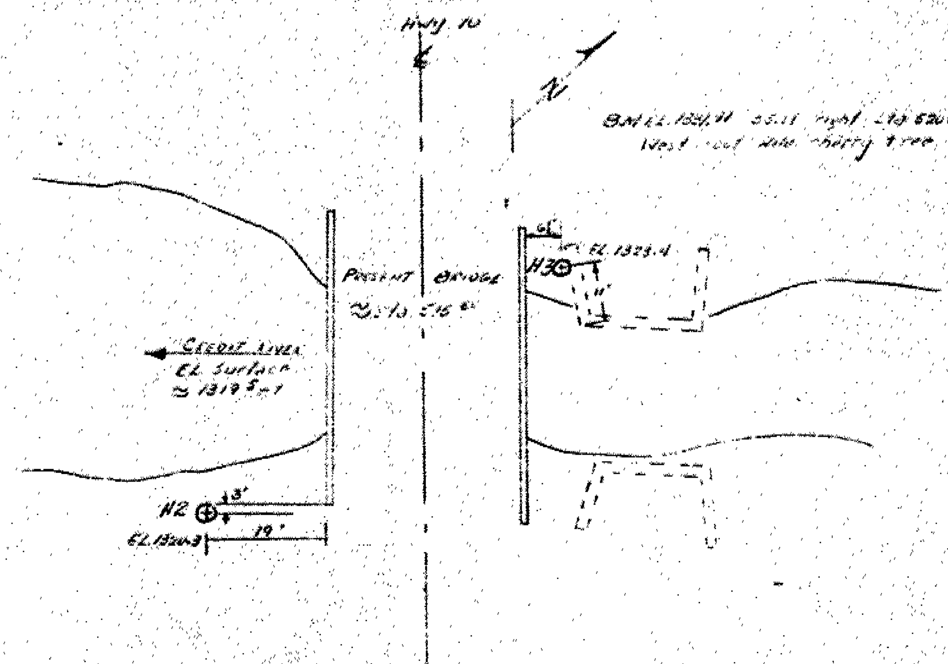
W.A.Trow,P.Eng.

REGISTERED PROFESSIONAL ENGINEER
CIVIL
W. A. TROW
PROVINCE OF ONTARIO

WAT/ND
In quadruplicate

Racey, MacCallum & Associates Ltd.

Order No. 3500/T 457
Prep. By M.T.



Sketch of Site Showing Location of Boreholes

Enclosure No. 1

Encl No. 2

Order No. 5500/7407 **RACEY, MacCALLUM AND ASSOCIATES**
LIMITED
Foundation Engineering Division

L.B.
Driller

Hole Begun ________

Helper

Hole Ended ________

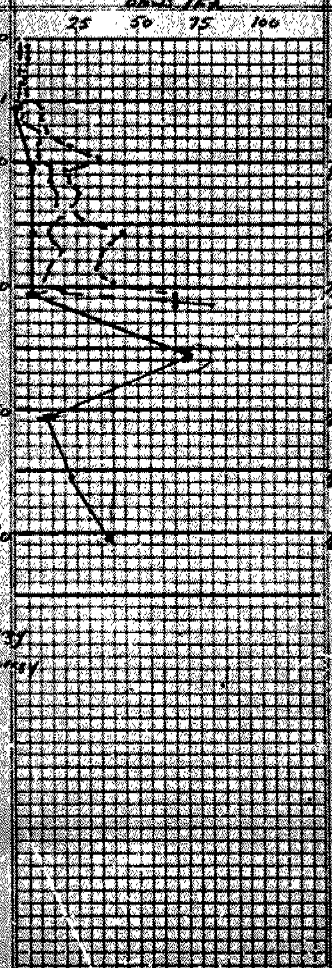
Engineering Data Sheet for Borehole: 2

Checked by

Job Name: Credit River Bridge ≈ Sta 515+50 Hwy 10
Job Located: Approx 4 miles North Caledon Ont
Hole Located: See encl No 1
Hole Elevation: 1320³ Datum: Geodetic El 1331.91

Day Month Year

Depth	El.	Thickness	Symbol	Description	Sampling Method
–	1320³			Ground Surface	
				Black spongy peat	
				ground water rose to over top of (?)	
6.5	1313⁸			Med. sand and medium to large rounded limestone gravel	SS 1
11	1309³				SS 2
				Uniform grey silt	SS 3
21'	1299³			Dense gravel and boulders	SS 4
					SS 5
28'	1292³			Uniform dense grey silt	SS 6
					SS 7
42'				End of Bore	SS 8



Legend
SS = 2" O.D. split spoon
——— Blows/ft 2" SS. 350 lb lbs energy
— — — Blows/ft 3" core 350 lb lbs energy
– – – – Blows/ft 3" casing

Encl. No. 3

Order No.: S2661T487 **RACEY, MacCALLUM AND ASSOCIATES**
LIMITED
Foundation Engineering Division

Hole Begun: ______ Driller: ______

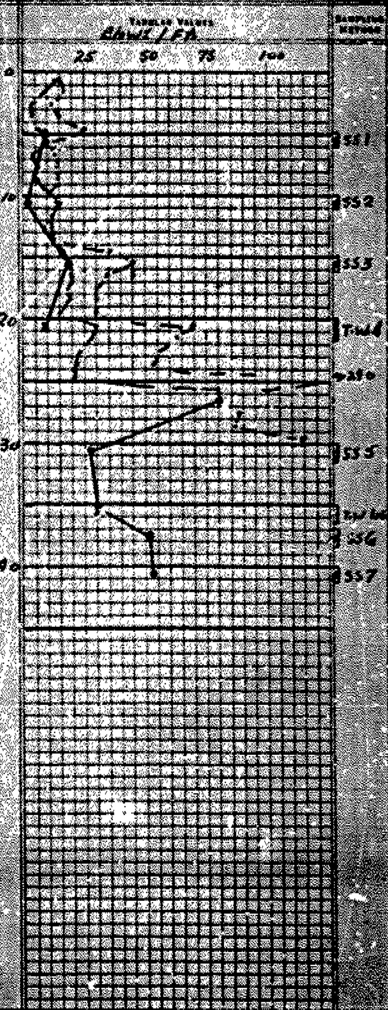
Hole Ended: ______ **Engineering Data Sheet for Borehole:** 3 Helper: ______

Job Name: Credit River Bridge ≈ Sta 515+50 Hwy. 10 Checked by: ______
Job Located: approx 4 mi North Caledon, Ont
Hole Located: See encl. No. 1
Hole Elevation: 1323.4 Datum: Geodetic El 1331.91

Day ______ Month ______ Year ______

DEPTH	EL.	THICKNESS	SYMBOL	DESCRIPTION
—	1323.4	—		Ground Surface
				Black sandy top soil
				Ground Water level at river level
12	1311.4			Med. dense coarse grey silt
24	1299.4			Rounded gravel + boulders
31	1292.4			Uniform dense grey silt, boulder at 35 ft
				End of bore

Legend
See encl. No. 2



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

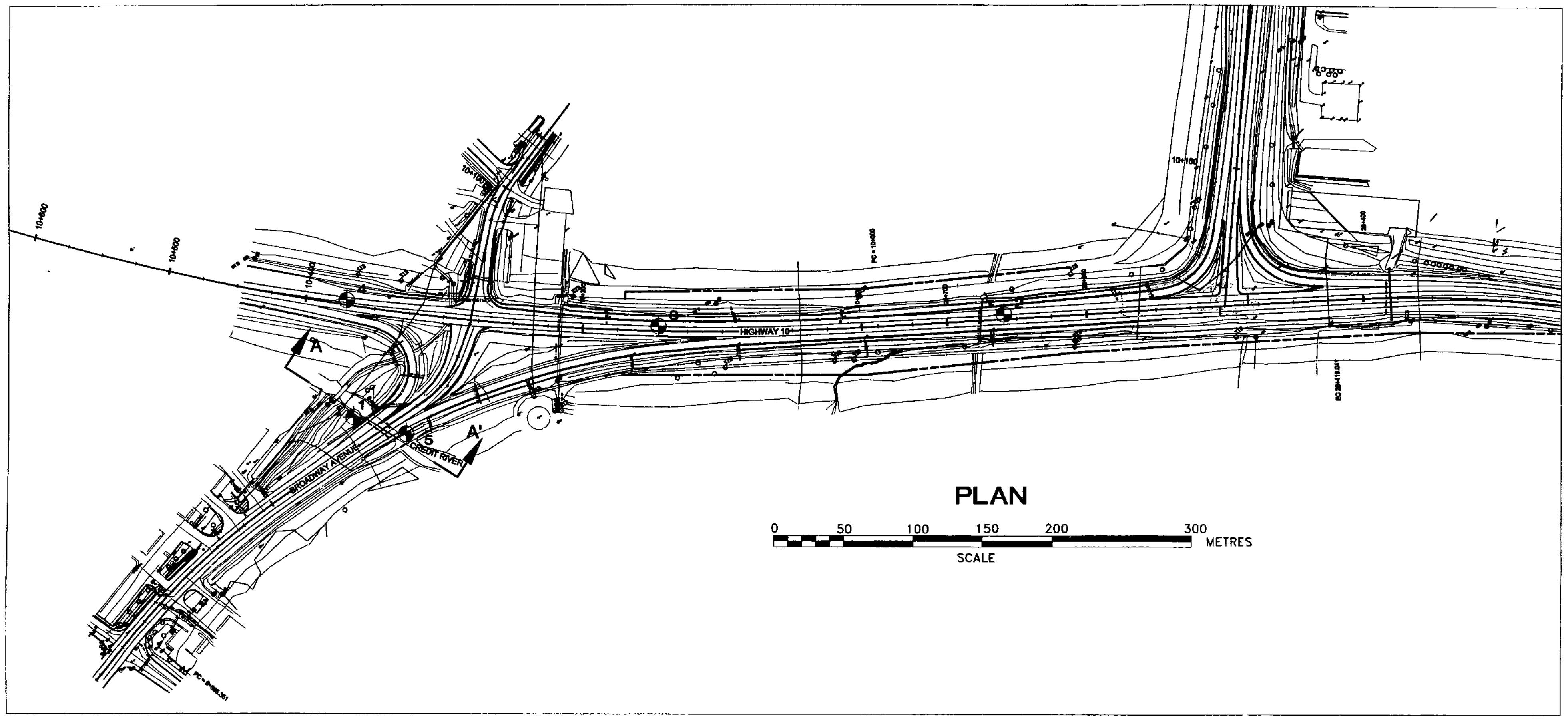
DIST 3 HWY 10
CONT. No.
WP No. 449-97-01

CREDIT RIVER CULVERT
BROADWAY AVENUE
BOREHOLE LOCATIONS & SOIL STRATA

SHEET

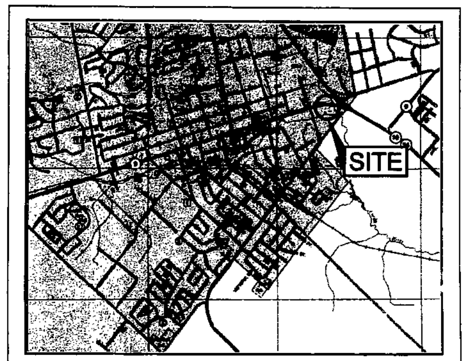
Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

SCALE 0 50 100 150 200 300 METRES



KEY PLAN

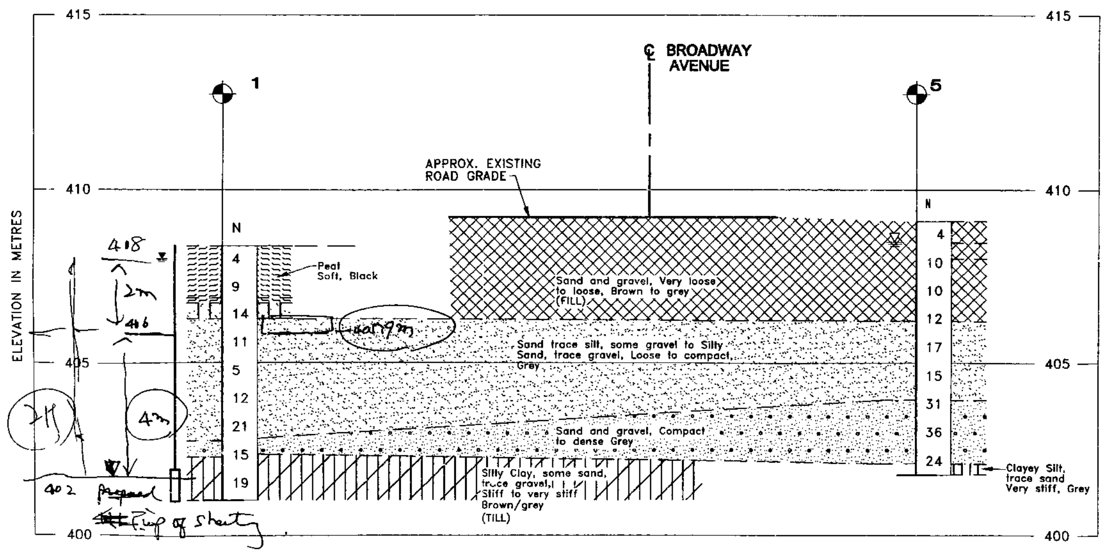
LEGEND	
	Borehole
	Seal
	Piezometer
N	Standard Penetration Test value
	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
	WL in piezometer on NOV. 5, 1999
	WL during drilling

No.	ELEVATION (m)	LOCATION NORTHING (m)	LOCATION EASTING (m)
1	408.38	4864820.44	257879.98
2	426.12	4864613.67	258304.35
3	413.99	4864748.42	258096.04
4	410.86	4864892.59	257924.09
5	409.10	4864788.84	257904.22

NOTES
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

※: check integrity of existing Foundation

Dewatering
1) Lower water level with well point or well point
2) prevent foundation disturbance



SECTION A-A'

HORIZONTAL SCALE 0 4 8 12 16 20 METRES

VERTICAL SCALE 0 2 4 6 8 10 METRES

1	28/01/00	ASP	FINAL
NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 10		PROJECT NO.: 991-1171	DIST. 3
SUBM'D. ASP	CHKD:	DATE: 1999 12 13	
DRAWN: JFC	CHKD. ASP	APPD.	DWG. 1

1" = 1" imp. (1:2,000ms)

01171001.DWG