G.I.-30 SEPT. 1976

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 316-190

DIST. 9 REGION

W.P. No. 145-74-04

CONT. No. 83-15

W. O. No.

STR. SITE No. 3-356

HWY. No. 16N

LOCATION Stevens Creek Bridge

No. of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

Sach

FOUNDATION INVESTIGATION REPORT

CONTRACT NO 83-15

Ministry of
Transportation and
Communications

Ontario

INDEX

Note: For purposes of this contract this report supercedes all other foundation reports prepared by or for the Ministry in connection with the above-mentioned project.

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1,%	POROSITY	I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1,%	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_U	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_P	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_S	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX $= w_L - w_P$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL				k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$	j	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL	e_{max}	1,%	VOID RATIO IN LOOSEST STATE			

FOUNDATION INVESTIGATION REPORT

For

W.P. 145-74-04
Structure Site 3-356
Proposed Crossing at Steven Creek and Hwy. 16N
District of (Ottawa), Eastern Region

INTRODUCTION

Warnock Hersey Professional Services Limited have been retained by the Ontario Ministry of Transportation and Communications to provide geotechnical engineering services in connection with the above project. The terms of reference were to carry out a site investigation for the design and construction of a new bridge at this site.

The field work carried out by the geotechnical consultants between 81 11 23, 24 consisted of the drilling of two sampled boreholes, one on each side of the creek, supplemented by two additional dynamic cone penetration tests. The investigation was carried out by means of a muskeg vehicle mounted continuous flight auger machine, equipped with hollow stem augers.

Additional fieldwork was undertaken on 82 12 07 by this Section and consisted of an additional sampled borehole (B.H. 101).

SITE DESCRIPTION AND GEOLOGY

The site is located in North Gower Township, some 35 kilometers south of Ottawa. Specifically, the proposed structure site is located 1.3 km north of Roger Steven Road (Cty Rd. 4), some 300 m west of 2nd Line Rd. At this location, Steven Creek is approximately 18 m wide and 3 m deep. The flow is in an easterly direction towards the Rideau River.

The area is designated physiographically as the North Gower Drumlin Field. Air photos of the area and ground observations show the presence of buried drumlins in the vicinity. These drumlins are oriented more or less north-south. A mantle of marine soil is draped over these drumlins and other similar glacial inorganic land forms.

The marine soil originated with the inundation of the area by the Champlain Sea. Therefore, silts and Leda clays are common in this area.

Due to the undulating nature of the topography, several springs have developed in the area. The spring waters originate in the higher ground where the mantle of marine soil is thin. The higher ground is generally a glacial landform. Springs are frequently found in the low ground between two relatively prominent glacial landforms, particularly where erosion has removed much of the cohesive marine soil mantle. Elsewhere, artesian groundwater conditions are generally encountered at depth.

At this site, the south approach of the proposed crossing consists of a cleared farmers field. The ground sits about 2 to 3 metres higher than the north approach, where the land is bush covered. A drainage tributary stream flows in from the north to join Steven Creek near the north approach area. Within the bush covered area, there are ox-bow shaped remnants of former stream tributary meanders. Both the north and south banks of the stream support swampy vegetation such as reed grass and sedges. There are no signs of slope instability, past or present, along the higher south bank of the creek near the proposed crossing.

SUBSURFACE CONDITIONS

General

The soil conditions across the site are fairly consistent. Briefly, the soil stratigraphy consists of a surficial organic layer followed by a highly plastic, sensitive clay, overlying a silt deposit.

The silt stratum, which contains clay seams and silty clay zones, is underlain by a very dense and gravel glacial till of undetermined thickness.

Reference should be made to the Record of Borehole Sheets contained in the Appendix of this report. These sheets contain the description and extent of the soil types encountered, and in summarized form, field and laboratory test results.

The stratigraphical profile shown on Drawing 2 is based on this information and shows the location and elevation of the borings.

A detailed description of the various strata are given below:

Surficial Organics

Along the higher flat-lying land on either side of the stream banks, the topsoil is about 200 mm in thickness. Along the shores of the river in the vicinity of the proposed crossing, the upper 1.5 metres consists of a plastic organic silt with some fine sand and clay. This material was fairly compressible but not spongy. It's moisture content was over 82 percent. An N value of 1 blow/0.3 m was recorded at one location in this deposit. It's consistency is described as soft.

Clay

Immediately below the topsoil on the flatter ground, or organic silts along the shores, there is present a clay deposit which is identified as Leda clay.

The thickness of this deposit varies depending on surface relief. At the south approach; it is approximately 7 metres thick whereas at the north approach it is only about 4 metres thick. In either case, however, the deposit extends down to about elevation 80.5.

The distinguishing characteristics are enumerated in Table 1 for the north and south approaches.

Table 1 Physical and Engineering Properties of "Undesiccated" Clay at Steven Creek

	North Approach		South Approach	
	Range	Mean	Range	Mean
Moisture Content (%)	74-85	78	56-75	65
Liquid Limit (%)	52-80	64	56-66	60
Plastic Limit (%)	28-31	30	24-29	27
Plasticity Index (%)	23-49	34	32-37	33
Liquidity Index	0.9-2.0	1.6	1.0-1.2	1.1
Unit Weight (kN/m^3)	15.2-15.6	15.3	15.5-16.8	16.2
Undrained Shear Strength (kPa)				
Field Vane	22-29	25	30-39	33
UU Triaxial	20-29	25	21-28	25
Lab Vanes- All values were lower than field or triaxial values				
Sensitivity	4-9	6	6-8	7

Preconsolidation, P_c, (kPa)	100-105
C_c	1.15
C_r	0.07
C_v (mm^2/sec)	0.5

The south approach ground surface sits some 2 to 2.5 metres higher than the north approach ground surface. Hence, at the south approach, the upper 2.5 metres or so of the clay has been desiccated and exhibits a natural moisture content in the range of 36 to 46 percent, with an average of 42. In this desiccated zone, the undrained shear strength measured by the UU Triaxial test ranged from 68 to 54 (kPa), decreasing rapidly with depth. Significant strength differences were found within each thin-walled tube near the transition zone from desiccated to undesiccated.

The soil in the desiccated zone has a mottled grey-brown colouring. Below the desiccated zone, the soil exhibits a fairly uniform grey colour with black spots and streaks representing fossil remnants now turned organic due to lack of oxygen in a reducing environment.

An examination of the soil properties in Table 1 shows some interesting differences. The mean moisture content of the clay below the desiccated zone is significantly lower than that where no desiccated crust is present. This difference is reflected also in the other index properties, including the unit weight.

The Atterberg limits of the clay are plotted on Figure 1. It can be seen that the plasticity of the soil increases with depth, each subsequent sample plotting above the last sample along the A-line.

There was a significant difference observed in the field vane shear strengths between the south and north approach clays. Where the desiccated zone is present, the field vanes indicate a substantially higher undrained shear strength than where the desiccated zone is absent. A similar difference was not noticed in the UU triaxial tests. The laboratory vane tests are not reported since the values measured in the field on the recovered samples were well below those measured by the field vane and in the triaxial test. This would indicate a significant influence of scale on the measured shear strengths. The laboratory vane used was a 6-bladed, commercial, pocket sized torvane.

It is reasonable to assume from the data given above that the removal of the desiccated zone at the north approach has affected to a considerable extent the physical properties of the clay. The removal of overburden (desiccated zone) by erosion has resulted in vertical stress relief followed by moisture intake and reduced shear strengths.

A sample of the clay from the borehole at the north approach was tested to determine its preconsolidation value. The result is shown on Figure 2. The e-log p curve in Figure 2 is typical of Leda clay soils. The preconsolidation is estimated at 100 to 105 kPa. Part of this high value is no doubt attributable to erosion of 2.5 to 3 metres of overburden. Assuming the overburden weighs 17 kN/m^3, the stress relief is in the order of 50 kPa. Therefore, only 50 kPa (100-50) represents preconsolidation due to either chemical bonding or aging of the clay. These relatively high values for this soil are confirmed by extensive testing carried out by the National Research Council of Canada for the instrumentation of the Rideau River Bridge at Kars, some 2.5 kilometers distant from this site. The NRC consolidation testing of Leda clay from Kars gave preconsolidation (p_c) values ranging from about 90 kPa (minimum estimate) to as high as 170 kPa (the most probable value being around 130 kPa). Therefore, the tested value of around 100 kPa fits within the NRC range of tested values.

Silt

At elevation 80.5, the clay deposit transitions into a silt deposit which extends to the glacial till stratum. The thickness of the silt deposit is about 6 metres. The deposit is non-homogeneous in that it contains zones of silty clay and distinct seam or bands of clay. In a typical sequence of stratification, the soil consists of a 100 mm layer of slightly plastic silt, then another layer of non-cohesive silt. With depth, the silt layers become thicker and less plastic; the frequency of occurrence of the thin clay bands or seams diminishes. In the last one metre or so of the deposit, some sand and gravel is present.

The silt layers are extremely dilatant. However, the clay seams or bands were found to be brittle and very stiff. The clay is highly plastic and exhibits a cubic fabric structure and fractures conchoidally. It's texture is smooth.

One careful separation of the two soil types was carried out to measure their moisture contents. The moisture content of the silt was measured to be 32 percent whereas that of the immediately adjoining clay seam was found to be 71 percent.

During deposition, the silt has become contaminated with clay and therefore exhibits some plasticity characteristics. The composite effect is a soil mixture having the plasticity characteristics of a silty clay, as shown on Figure 1. Vane tests were taken in this deposit in the belief that it was a plastic soil. However, careful examination of the extruded tube samples reveals the soil to be a silt; therefore, the vane shear strength values may not be an appropriate measure of the strength of the silt.

The physical properties of the silt deposit are summarized in Table 2.

Table 2 Physical Properties of Silt Deposit at Steven Creek

Physical Property	Siltier Portion		Clay Portion	
	Range	Ave.	Range	Ave.
Moisture Content (%)	21-32	26	32-71	45
Liquid Limit (%)	N.P.	-	29-41	35
Plastic Limit (%)	N.P.	-	19-21	20
Plasticity Index (%)	-	-	10-20	15
Undrained Shear Strength (kPa)				
Field Vane	Range: 22-42			
UU Triaxial	Range: 18-35			
Unit Weight (kN/m^3)	Range: 16.6-19.2			

Based on "N" values of 1 to 2 blows/0.3 m and the low undrained shear strengths, the relative density of the silt is described as very loose whereas its consistency is described as soft in the more plastic zones.

Sand and Gravel (Glacial Till)

Below about elevation 75, the silt deposit is underlain by a glacial till consisting essentially of a mixture of sand and gravel with a trace of silt and clay. A grain size distribution curve is shown in Figure 3. The sand and gravel particles are angular.

The predominant constituent is a dark grey limestone which indicates the till to be a basal till derived largely from the local bedrock in the area. The material at first glance looks like weathered bedrock. However, there are also present, in the sand-sized grains, orthoclase feldspar, quartz and calcite components indicating the material to be of glacial origin and not a product of in-situ weathering.

N values in this till deposit ranged from 5 to 100 blows/0.3 m. Two dynamic cone penetration tests met refusal within this deposit. On the basis of these tests, the relative density of the sand and gravel is considered to be very dense. The upper 0.3 to 1.0 metres or so of the deposit has been influenced by contamination with the overlying silt as well as by the presence of artesian flow conditions. Therefore, it exhibits a lower relative density (N value of 5 blows). From the results of other soil investigations in the vicinity, it is believed that bedrock may be located not too far below the termination of the boreholes and cone tests within this deposit.

Auger refusal on what are probable boulders in the glacial till stratum was encountered at about 14.1 m depth (elevation 77.1 m) at B.H. 101, and at approximately 13.0 m depth (elevation 72.8) at B.H. 1.

GROUNDWATER CONDITIONS

The groundwater at B.H. 1 and B.H. 102 is located at about creek level. At the south approach (B.H. 4), the groundwater was about 1 metre below the ground surface. Therefore, the phreatic surface is presumed to follow the ground profile. Fluctuations of the groundwater level in the clay are likely to be minimal when the creek floods. From the desiccation observed in the clay deposit at the south approach, the long-term water level is assumed to prevail at a depth of about 2 metres.

Within the underlying glacial till, artesian flows were encountered on both sides of Steven Creek. Estimated pressure heads were 3 m and 2 m above the prevailing creek level. A very minor artesian condition was noted at Borehole 4. It is believed that the higher ground on the south shore is characterized probably by sub-artesian conditions.

K. Chak
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
CIVIL
M. DEVATA
PROVINCE OF ONTARIO

M. Devata
Senior Foundations Engineer

A P P E N D I X

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 1 — METRIC

W P 145-74-04 LOCATION STA 19+006.0 O/s 9.0m Rt of ℄ Hwy 16N ORIGINATED BY CM

DIST 9 HWY 16 N BOREHOLE TYPE Hollow Stem Auger COMPILED BY BD

DATUM Geodetic DATE 1981 11 23 CHECKED BY CM

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	'N' VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Artesian Head Approx. 3 m Above Ground						
85.8	Ground Surface						
0.0	TOPSOIL and ORGANIC SILT with Clay and Sand Soft	1	SS	1			82%
84.4 / 1.4	CLAY, Sensitive Grey Soft to Firm with trace of black Organic inclusions	2	TW	PM	84	15.2	
		3	TW	PM	82	15.1 / 15.4	85%; e_o = 2.023, C_c = 1.15, P_o = 18 kPa
		4	TW	PM		15.6	
80.3 / 5.5	SILT, Very Loose (or Soft where Cohesive) with periodic seams of Very Stiff Clay decreasing in frequency with depth	5	TW	PM	80		
		6	TW	PH	78	17.2	
		7	TW	PH		16.6	
		8	SS	2	76		
	Artesian Condition Encountered	9	TW	PM			
		10	SS	15	74		
74.5 / 11.3	GRAVELLY SAND (Glacial Till) Compact to Very Dense	11	SS	74			40 55 (5)
72.8 / 13.0	END OF BOREHOLE Refusal to Augering						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 10 20 30 40 50

PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%): 20 40 60

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 -◇- 5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 2

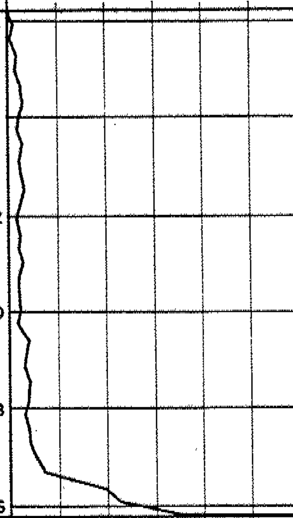
METRIC

W P 145-74-04 LOCATION STA. 19+024.0 O/s 6.0 m Lt. of ℄ Hwy 16 N ORIGINATED BY CM

DIST 9 HWY 16 N BOREHOLE TYPE Cone Penetration Test COMPILED BY BD

DATUM Geodetic DATE 1981 11 23 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
86.2	Ground Surface										
0.0	Probably CLAY						86, 84, 82, 80				
79.7 / 6.5	Probably SILT						78, 76				
75.8 / 10.4	END OF CONE TEST										



$+^3$, $\times^5$: Numbers refer to Sensitivity

15 ⟡ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 3 — METRIC

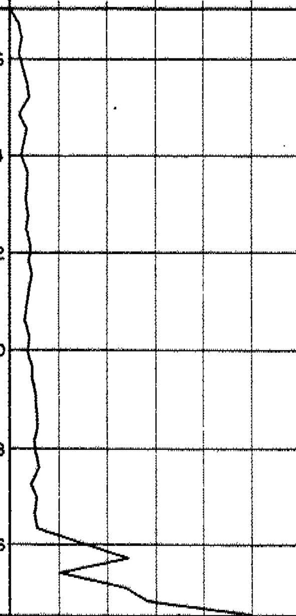
16

W P 145-74-04 — LOCATION STA. 18+973.0 o/s 5.0m Rt ℄ Hwy 16N — ORIGINATED BY CM

DIST 9 HWY 16 N — BOREHOLE TYPE Cone Penetration Test — COMPILED BY BD

DATUM Geodetic — DATE 1981 11 23 — CHECKED BY CM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
87.0	Ground Surface										
0.0	Probably Desiccated						86				
84.8 / 2.2	Probably CLAY						84, 82				
80.0 / 7.0	Probably SILT						80, 78, 76				
74.5 / 12.5	END OF CONE TEST										



+3, x5 : Numbers refer to Sensitivity — 15 ✧ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation and Communications
Ontario

17

RECORD OF BOREHOLE No 4

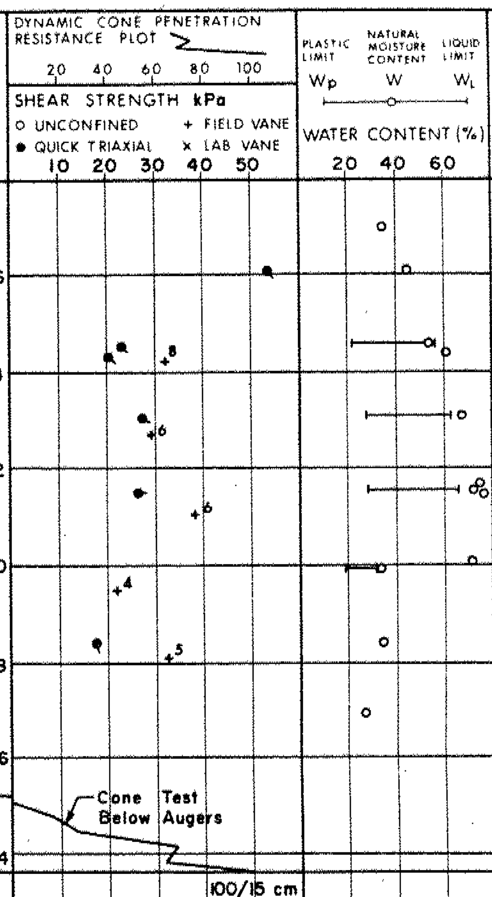
METRIC

W P 145-74-04 LOCATION STA. 18+965.0 O/S 5.0 m Lt of ℄ Hwy 16 N ORIGINATED BY CM

DIST 9 HWY 16N BOREHOLE TYPE Hollow Stem Auger and Cone Test COMPILED BY BD

DATUM Geodetic DATE 1981 11 24 CHECKED BY CM

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³
87.9	Ground Surface					
87.3 / 0.0	TOPSOIL					
0.6	CLAY, Desiccated Stiff Mottled Brown	1	SS	7	▽	
		2	TW	PH		16.8
84.8 / 3.1	CLAY, Sensitive Grey Soft to Firm with trace of black Organic inclusions	3	TW	PM		16.8
		4	TW	PM		16.4
		5	TW	PM		15.5
80.5 / 7.4	SILT, Very Loose (or Soft where Cohesive) with periodic seams of Very Stiff Clay becoming less frequent with depth	6	TW	PM		
		7	TW	PM		19.1
		8	SS	1		
75.3 / 12.6	END OF BOREHOLE	9	SS	6		
74.4 / 13.5	Probably SILT					
73.7 / 14.2	Probably SAND and GRAV. (Till)					
	END OF CONE TEST					



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

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 -ф- 5 (20 / 10) (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 101

METRIC 18

W P 145-74-04 LOCATION Sta. 18 + 977.0; O/S 6.0 m Lt. ℄ Hwy. 16N ORIGINATED BY KC

DIST 9 HWY 16N BOREHOLE TYPE Hollow Stem Auger COMPILED BY KC

DATUM Geodetic DATE 82 12 07 CHECKED BY [signature]

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS / REMARKS
					Artesian Head Approx. 2 m Above Ground
86.2	Ground Surface				
0.0	Topsoil & Organic Silt Soft	1	SS	3	
85.3 / 0.9	Clay, Sensitive Grey Soft to Firm with Trace of Black Organic Inclusions	2	TW	PH	
		3	TW	PM	
80.4 / 5.8	Silt, Very Loose (or Soft to Firm where Cohesive) with periodic seams of Very Stiff Clay decreasing in frequency with depth	4	TW	PH	
75.8 / 10.4	Sand and Gravel (Glacial Till) Compact to Very Dense	5	SS	5	Artesian Encountered
		6	SS	28	
		7	SS	100/8 cm	
72.1 / 14.1	End of Borehole Refusal to Auger Probable Boulders				

ELEVATION SCALE: 86, 84, 82, 80, 78, 76, 74

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 10 20 30 40 50

Field vane sensitivities plotted: 12 (approx. 83.8), 8 (approx. 80.7), 6 (approx. 77.8)

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

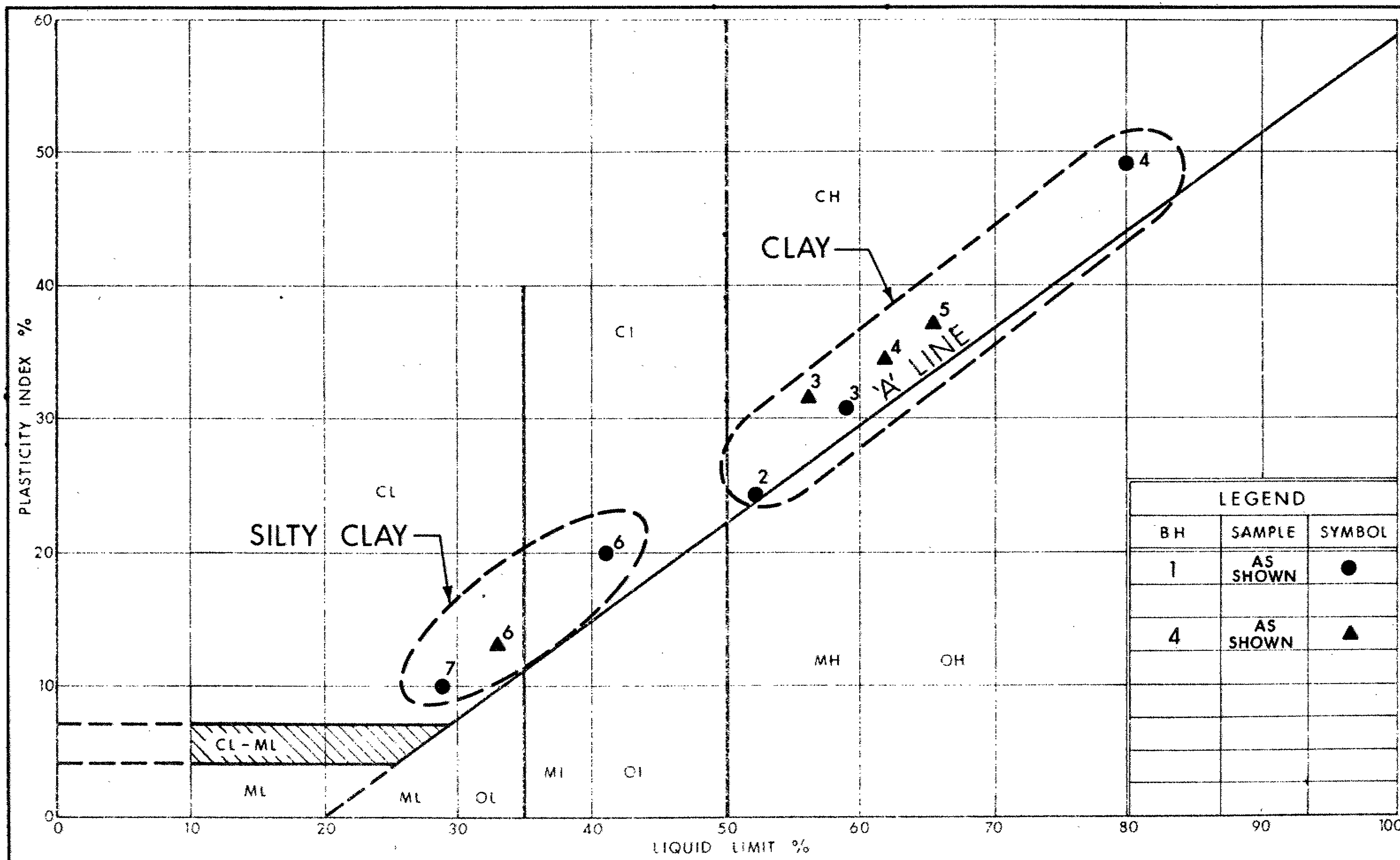
UNIT WEIGHT γ

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

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION



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Ontario

PLASTICITY CHART

FIG No 1

W P 145-74-04

Steven Creek

VOID RATIO–PRESSURE DIAGRAM
CONSOLIDATION TEST

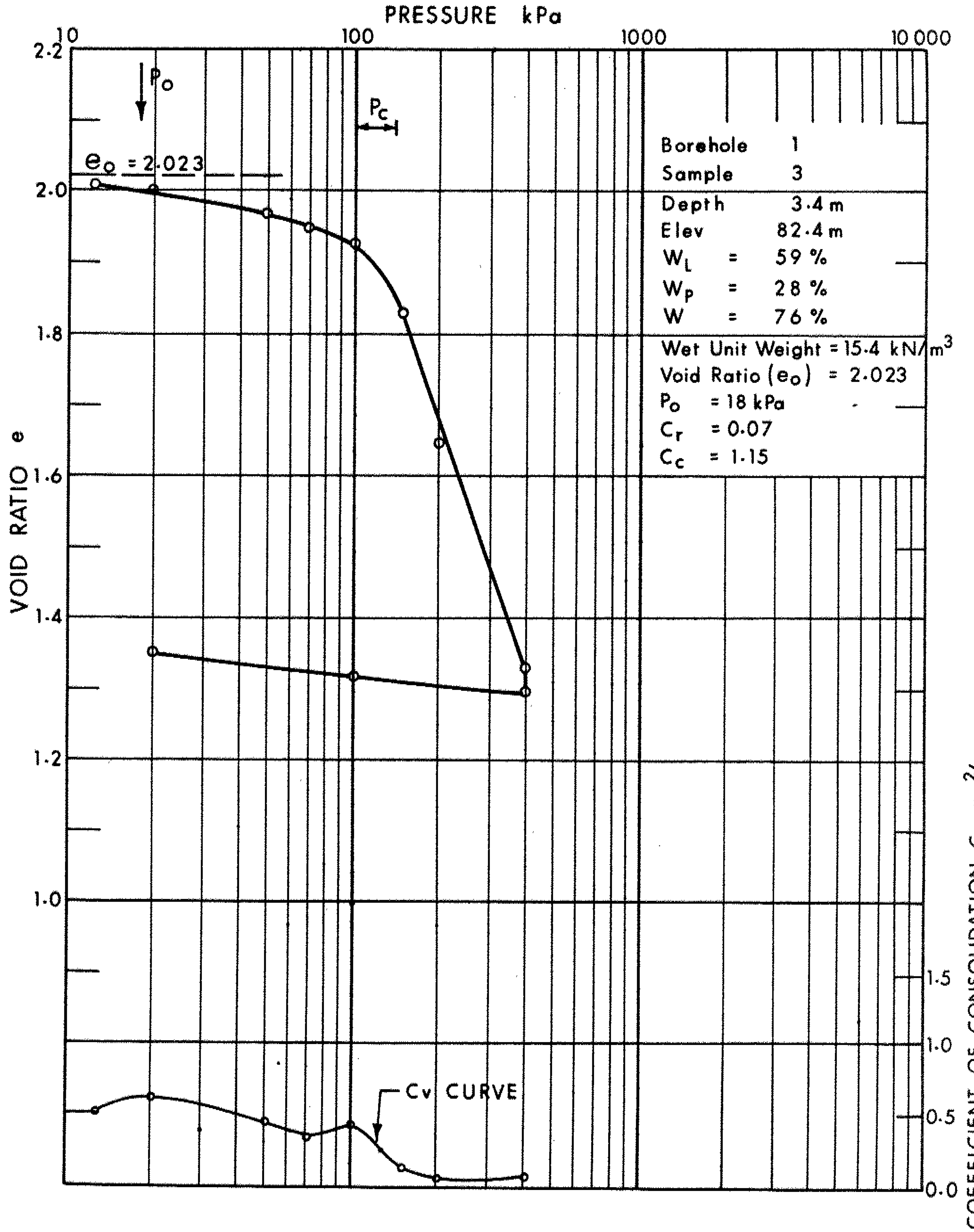
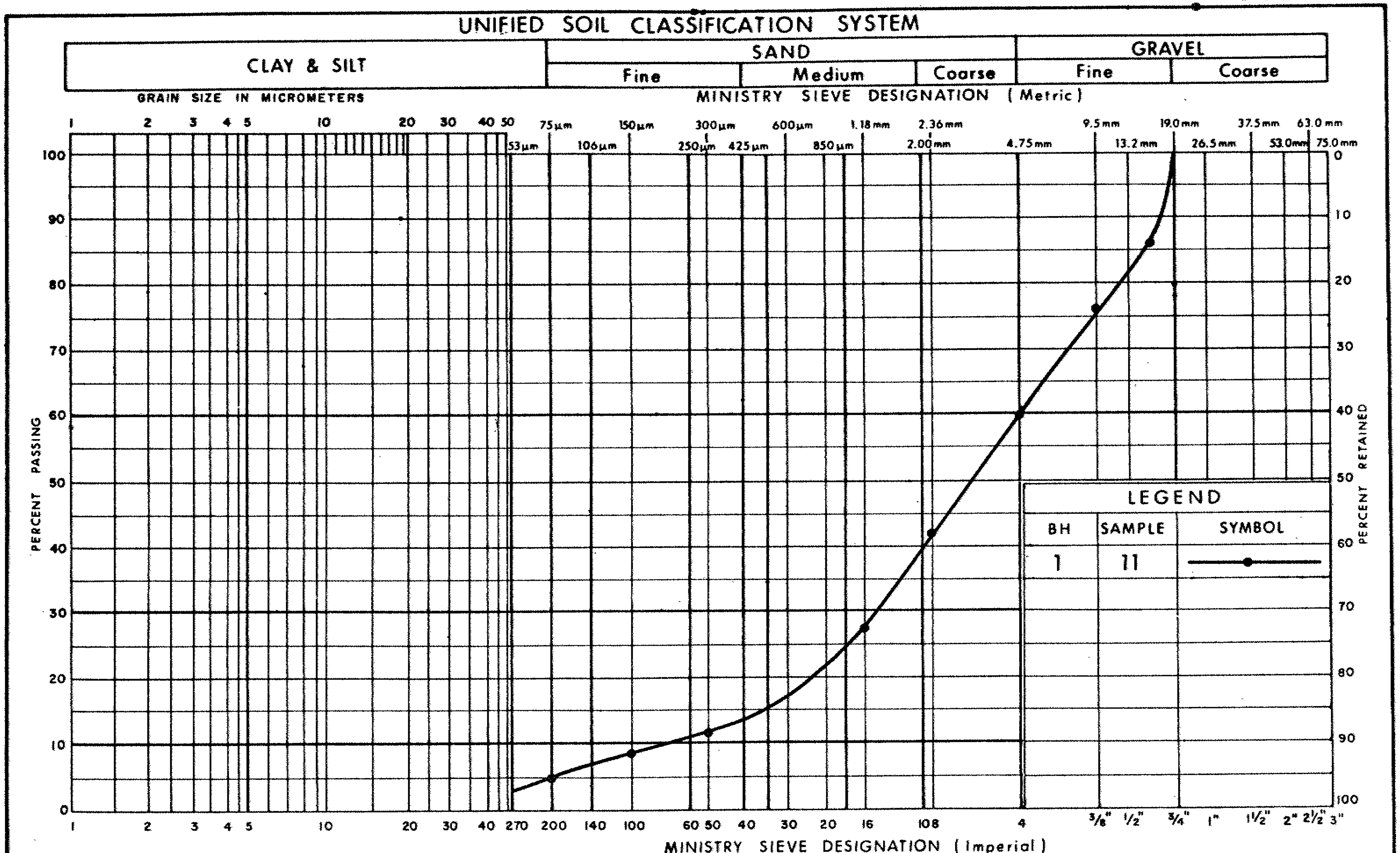


FIG No 2

WP 145–74–04
Steven Creek

78 12 M



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GRAIN SIZE DISTRIBUTION
SAND AND GRAVEL (Glacial Till)

FIG No 3
W P 145-74-04
STEVEN CREEK

FOUNDATION INVESTIGATION REPORT

for

W.P. 145-74-05
Structure Site 3-358
Proposed Crossing at Mud Creek and Hwy. 16N
District 9, Ottawa

INTRODUCTION

Warnock Hersey Professional Services Limited had been retained by the Ontario Ministry of Transportation and Communications to provide a foundation investigation in connection with the above project.

The field work carried out by the geotechnical consultants between 81 11 24, 25 consisting of the drilling of two sampled boreholes, one on each side of the creek, supplemented by two additional dynamic cone penetration tests and one continuous vane shear strength hole. The investigation was carried out by means of a muskeg vehicle mounted continuous flight auger machine, equipped with hollow stem augers.

Additional field work was carried out on 82 12 08 (B.H. 101 & 102) by this Office, and consisted of an additional 2 sampled boreholes.

DESCRIPTION OF SITE AND GEOLOGY

The site is located in North Gower Township, some 35 kilometers south of Ottawa. Specifically, the proposed structure site is located off 2nd Line Road between existing Hwy. 16 and Carsonby. At this location, Mud Creek is approximately 5 to 7 m wide and about 1 m deep. The flow is in a northerly direction.

The area is designated physiographically as the North Gower Drumlin Field. Air photos of the area and ground observations show the presence of buried drumlins in the vicinity. These drumlins are oriented more or less north-south. A mantle of marine soil is draped over these drumlins and other similar glacial inorganic land forms.

The marine soil originated with the inundation of the area by the Chaplain Sea. Therefore silts and Leda clays are common in this area.

At this site, Mud Creek meanders on a relatively flat-lying plain bordered in places by glacial landforms such as moraine remnants. The proposed alignment of Hwy. 16N runs parallel to and about 50 m west of existing 2nd Line Road. This township road crosses Mud Creek with a single span steel beam structure (structure site 3-143). At the south approach bearing ledge, the concrete beneath the steel beam bearing plates is cracked in a semicircular fashion, and, at one corner, the concrete has spalled, exposing the reinforcing. The approaches to this structure are about 3 metres in height and show no sign of excessive or differential settlement, nor instability.

SUBSURFACE CONDITIONS

The soil conditions across the site are fairly consistent. Briefly, the soil stratigraphy consists of a surficial slightly organic layer followed by a highly plastic, sensitive clay, overlying a silt deposit. The silt stratum, which contains clay seams and silty clay zones is underlain by a glacial till stratum of undetermined thickness, comprised either of very dense sand and gravel or hard silty clay.

Auger refusal on what are probably boulders in the glacial till stratum was encountered at about 13.2 metre depth (elevation 77.2) in Borehole 101, and at approximately 12.0 metre depth (elevation 78.0) in Borehole 102.

Reference should be made to the Record of Borehole Sheets contained in the Appendix of this report. These sheets contain the description and extent of the soil types encountered, and in summarized form, field and laboratory test results. The stratigraphical profile shown on Drawing 3 is based on this information and shows the location and elevation of the borings.

A detailed description of the various strata are given below:

Surficial Soils

Surficial soils consist mainly of topsoil and some organically contaminated sandy silt. The maximum depth of this material was 1.5 metres at B.H. 101. The material is not highly compressible and exhibits a moisture content of about 25 percent. One N value in this material was 8 blows/0.3 m indicating it to be of loose compactness.

Clay

Underlying the topsoil, or the organically contaminated sandy silt, there is a 2.5 to 5 metre thick deposit of clay, the upper 2.5 metres of which is desiccated. The bottom of the clay stratum is located about elevation 85.

The upper desiccated zone exhibits a characteristic mottled brown colouring and contains root hairs. Due to the fissured nature of the desiccated zone, samples could not be suitably trimmed for triaxial testing. However, field vane and undrained strength values exceeded 50 kPa, indicating this material to be of generally stiff consistency. N values of 15 and 8 blows/0.3 m were recorded in this zone.

Below the desiccated zone, the clay exhibits a characteristic grey colour, with a trace of black mottling typical of Leda clays. The clay is homogeneous in texture and sensitive to remoulding. Typical physical properties of the clay are shown in Table 1.

Table 1 Physical Properties of Clay at Mud Creek Site

	Range	Average
Moisture Content (%)	82-85	83
Liquid Limit (%)	50-75	65
Plastic Limit (%)	22-31	27
Plasticity Index (%)	32-46	38
Liquidity Index		1.4
Field (Vane) Undrained Shear Strength (kPa)	31-57	37

The Atterberg limits are plotted on Figure 1 and show the material to be highly plastic.

Based on the field vane tests, particularly the continuous vanes at Borehole 3, the consistency of the clay may be described as firm to stiff, based on an average undrained shear strength of about 37 kPa.

Some hand probings were made in this deposit at the bottom of Mud Creek. These probings show the upper one metre of the soil to be quite soft, (estimated shear strengths in the order of 10 kPa).

Silt

At about elevation 85±, the clay is underlain by a silt deposit which extends for a thickness of 3.5 to 6 metres to a till stratum.

The silt is grey in colour and extremely dilatant. One grain size distrubution curve is shown on Figure 2. The silt has trace amounts of fine sand and clay.

Field vane tests were carried out in this deposit in the belief that the material was cohesive. It was believed that the predominant deposit was clay, with silt zones. In fact, the opposite was revealed when all the recovered soil samples were examined in the laboratory and tested for verification of classification.

At the clay-silt strata interface, the silt exhibits slight plasticity and in fact behaves as a silty clay of low plasticity would behave. With increasing depth, however, the silt character dominates the profile.

The distinguishing characteristic of this deposit is the presence of clay bands or seams at periodic intervals. The clay bands are typically about 5 to 10 mm in thickness. The clay is very stiff in consistency, smooth in fabric and cubically fractured in structure. It is extremely brittle.

The natural moisture content of the silt ranged from 23 to 29 percent, with an average of 26 percent. The moisture content of a clay seam from this silt was measured to be 74 percent.

Based on N values of 1 blow/0.3 m, the silt is considered to be in a very loose state of density.

Glacial Till

The silt deposit is underlain by a glacial till stratum of undetermined thickness comprised either of sand and gravel or silty clay, sand and gravel. From other soils investigations in this area, it is believed the till deposit is fairly extensive in depth.

A typical grain size distribution of the sand and gravel portion of this deposit is shown on Figure 3. The material is fairly coarse but does contain trace amounts of fines. The sand and gravel sizes are sharp and angular and consist of orthoclase feldspars, quartz, calcite and limestone particles. Hence, the glacial till is basal, derived from the local rock in the area.

Based on N values of 69 to 100 blows/0.3 m and results of the cone tests, the glacial till deposit is considered to be very dense within the sand and gravel portion with locally loose to compact zones, and hard within the silty clay portion of the till.

Borehole 101 encountered auger refusal on probable boulders within the till at about 13.2 metre depth (elevation 77.2). Borehole 102 met refusal to augering at 12.0 metre depth (elevation 78).

Groundwater

Groundwater levels at the site were found to correspond closely to the water level of the adjacent Mud Creek. However, at B.H. 102, groundwater was encountered once the borehole penetrated the sand and gravel, and appeared to stabilize at approximately creek level. In addition, at B.H. 1 south of Mud Creek, an artesian condition was noted upon penetration of the sand and gravel layer, and was estimated at approximately 2 metres above ground surface.

K.D. Chak
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
CIVIL
M. DEVATA
PROVINCE OF ONTARIO

M. Devata, P. Eng.
Senior Foundations Engineer

83 03 18

A P P E N D I X

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Ontario

RECORD OF BOREHOLE No 1 — METRIC

W P 145-74-05 | LOCATION STA 22+965.0 O/S 10.0m Rt of ℄ Hwy 16 N | ORIGINATED BY CM

DIST 9 HWY 16N | BOREHOLE TYPE Hollow Stem Auger and Cone Test | COMPILED BY BD

DATUM Geodetic | DATE 1981 11 24 and 25 | CHECKED BY CM

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
89.4	Ground Surface				
0.0	TOPSOIL and Fine SANDY SILT Slightly Organic, trace of Clay	1	SS	8	
88.0 / 1.4	Desiccated Stiff to Firm	2	SS	3	
87.0 / 2.4	CLAY, Sensitive Firm to Stiff Grey	3	TW	PM	
85.4 / 4.0	SILT, Very Loose with thin Very Stiff Clay bands at 30 to 70 mm intervals Becoming Soft cohesive Silt with depth	4	TW	PM	
		5	TW	PH	0 8 (92)
		6	SS	1	
		7	SS	1	
79.7 / 9.7	SAND AND GRAVEL with some Silt and Clay (Glacial Till) Very Dense	8	SS	96	48 46 (6)
		9	SS	69	
76.8					
76.3	END OF BOREHOLE				
13.1	END OF CONE TEST				

Ground water conditions: Artesian Head Approx. 2m Above Ground; Artesian condition Encountered

Elevation scale: 88, 86, 84, 82, 80, 78

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 10 20 30 40 50

Field vane values (sensitivity): +6, +5, +4, +4

100/25 cm
Cone Refusal at 13.1m depth

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; 81.7%

UNIT WEIGHT γ

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

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Ontario

RECORD OF BOREHOLE No 2

METRIC

W P 145 - 74 - 05 LOCATION STA 9+971.8 O/S 22.5 m Rt ℄ Creek Diversion ORIGINATED BY C M

DIST 9 HWY 16 N BOREHOLE TYPE Hollow Stem Auger and Cone Test COMPILED BY B D

DATUM Geodetic DATE 1981 11 25 CHECKED BY C M

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
90.2	Ground Surface					
0.0 / 89.6	TOPSOIL				*	90
0.6	Desiccated, Stiff Mottled Brown - Grey	1	SS	15		
		2	SS	6		
87.7 / 2.5	CLAY, Sensitive Firm to Stiff Grey					88
		3	SS	1		
85.6 / 4.6	SILT, Very Loose with thin Clay bands at 70 mm intervals Becoming Soft cohesive Silt with depth	4	TW	PH		86
		5	TW	PM		84
		6	TW	PM		
82.0		6A	SS	PH		
8.2	END OF BOREHOLE Probably SILT					
80.5 / 9.7	Probably SAND and GRAVEL (Glacial Till)					80
79.4 / 10.8	END OF CONE TEST					

* Water Level Not Established

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 10 20 30 40 50

Field vane sensitivities plotted: +6, +6, +3

Cone Test Below Augers

100/15 cm

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

WATER CONTENT (%): 20 40 60

84.8%

UNIT WEIGHT γ

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

$+^3$, x^5 : Numbers refer to Sensitivity

$15 \diamond 5$ (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 3

METRIC

31

W P 145-74-05 LOCATION STA 22 + 987.0 O/s 3.0 m Lt of ℄ Hwy 16N ORIGINATED BY C.M.

DIST 9 HWY 16 N BOREHOLE TYPE Augers ; Continuous Vanes and Cone Test COMPILED BY B.D.

DATUM Geodetic DATE 1981 11 25 CHECKED BY C.M.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 10 20 30 40 50	PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L / WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
90.2	Ground Surface										
0.0	Desiccated Very Stiff					▽	90 / 88				
87.0 / 3.2	CLAY, Sensitive Firm to Stiff Grey						86				
85.2 / 5.0	SILT, Probably banded with Clay Seams										
83.8 / 6.4	END OF VANE TEST						84				
	Probably SILT						82	Cone Test Below Augers			
81.1 / 9.1	Probably SAND and GRAVEL (Glacial Till)										
79.8 / 10.4	END OF CONE TEST						80	50/5cm and Bounding			

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation and Communications Ontario

RECORD OF BOREHOLE No 4

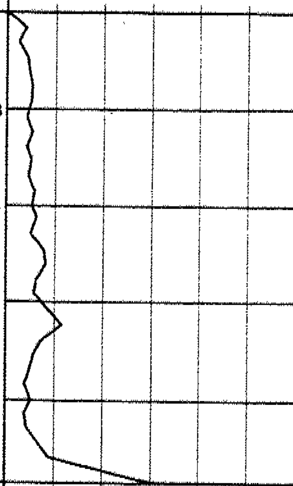
METRIC 32

W P 145-74-05 LOCATION STA 9+922.5 O/s 9.8 m Rt ℄ Creek Diversion ORIGINATED BY C M

DIST 9 HWY 16 N BOREHOLE TYPE Cone Penetration Test COMPILED BY B D

DATUM Geodetic DATE 1981 11 25 CHECKED BY C M

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH: o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
90.0	Ground Surface										
0.0	Probably CLAY						88				
							86				
85.3 / 4.7	Probably SILT						84				
							82				
81.5 / 8.5	Probably SAND and GRAVEL (Glacial Till)										
80.3 / 9.7	END OF CONE TEST							60/23 cm and Bouncing			



$+^3, \times^5$: Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

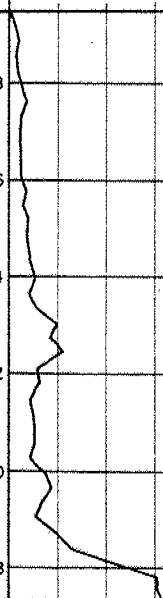
RECORD OF BOREHOLE No 5 — METRIC

W P 145 - 74 - 05 LOCATION STA 22+949.0 O/s 5.5 m Rt of ℄ Hwy 16 N ORIGINATED BY C M

DIST 9 HWY 16 N BOREHOLE TYPE Cone Penetration Test COMPILED BY B D

DATUM Geodetic DATE 1981 11 25 CHECKED BY C M

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
89.5	Ground Surface										
0.0	Probably CLAY						88, 86				
84.7 / 4.8	Probably SILT						84, 82, 80				
79.0 / 6.5	Probably SAND and GRAVEL (Glacial Till)						78				
77.4 / 12.1	END OF CONE TEST							63/20 cm and Bouncing			



$+^3$, $\times^5$: Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 101

METRIC 34

W P 145-74-05 LOCATION Sta. 23 + 005.0 o/s 5.5 m Lt. ℄ Hwy. 16N ORIGINATED BY KC

DIST 9 HWY 16N BOREHOLE TYPE Hollow Stem Auger COMPILED BY KC

DATUM Geodetic DATE 82 12 08 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
90.4	Ground Surface						
0.0	Desiccated Stiff to Firm					90	
		1	TW	PH			
88.0 / 2.4	Clay, Sensitive Firm Grey				▽	88	
		2	TW	PM			
86.4 / 4.0	Silt, Very Loose with thin Very Stiff Clay bands at 30 mm to 70 mm intervals Becoming Soft cohesive Silt with depth					86	
		3	TW	PH			
		4	TW	PH		84	
		5	SS	2		82	
	increasing Sand & Gravel	6	SS	3			
80.0 / 10.4	Silty Clay of low plasticity with Sand, trace of Gravel (Glacial Till) Hard Grey					80	
		7	SS	100/ 15 cm			
		8	SS	100/ 18 cm		78	
77.2 / 13.2	End of Borehole Refusal to Augering Probable Boulders						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%)

UNIT WEIGHT γ

Shear strength plot: + 7 (approx. 88 m, ~80 kPa), ● (approx. 88.5 m, ~80 kPa), + 9 (approx. 86.5 m, ~40 kPa), + 8 (approx. 85 m, ~40 kPa)

$+^3, \times^5$: Numbers refer to Sensitivity

15 -◇- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 102

METRIC

35

W P 145-74-05 LOCATION Sta. 22 + 988.0 o/s 5.5 m Rt. ₵ Hwy. 16N ORIGINATED BY KC

DIST 9 HWY 16N BOREHOLE TYPE Hollow Stem Auger COMPILED BY KC

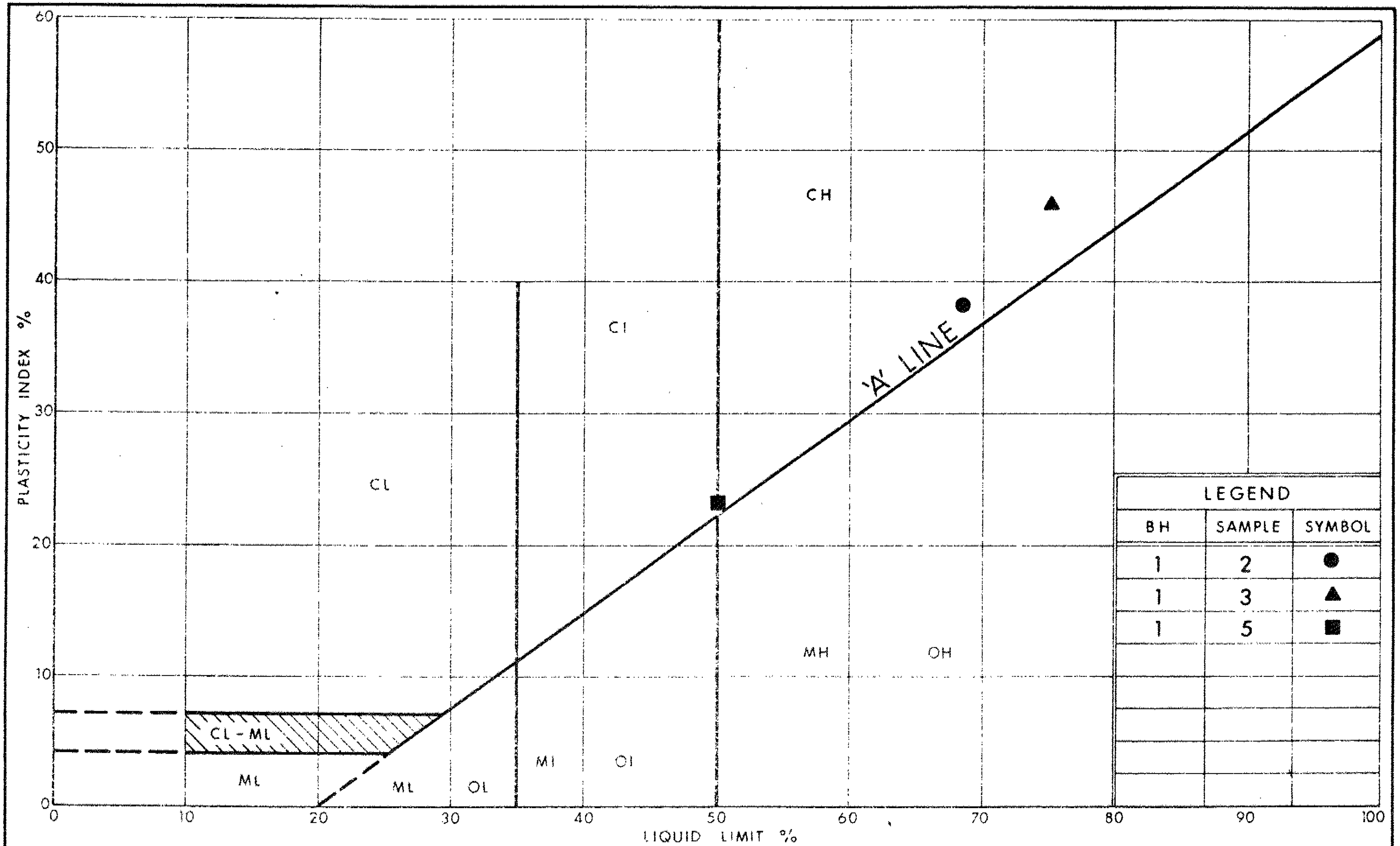
DATUM Geodetic DATE 82 12 08 CHECKED BY

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
90.0	Ground Surface										
0.0	Topsoil and Fine Sandy Silt, slightly*		1	SS	6						
89.1			2	SS	10						
0.9	Desiccated Stiff to Firm						88	+7			
87.6 / 2.4	Clay, Sensitive Firm Grey		3	SS	2	▽					
85.3							86				
4.7	Silt, Very Loose with thin Very Stiff Clay Bands at 30 mm to 70 mm intervals Becoming Soft cohesive Silt with Depth						84				
			4	SS	2		82				
	Increasing Sand & Gravel		5	SS	9		80				
79.9 / 10.1	Sand & Gravel (Glacial Till) Compact to Very Dense		6	SS	27						
78.0 / 12.0	End of Borehole Refusal to Augering Probable Boulders										

*organic
Trace of Clay

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 ⟡ 5 (%) STRAIN AT FAILURE (20 above, 10 below)



LEGEND

BH	SAMPLE	SYMBOL
1	2	●
1	3	▲
1	5	■

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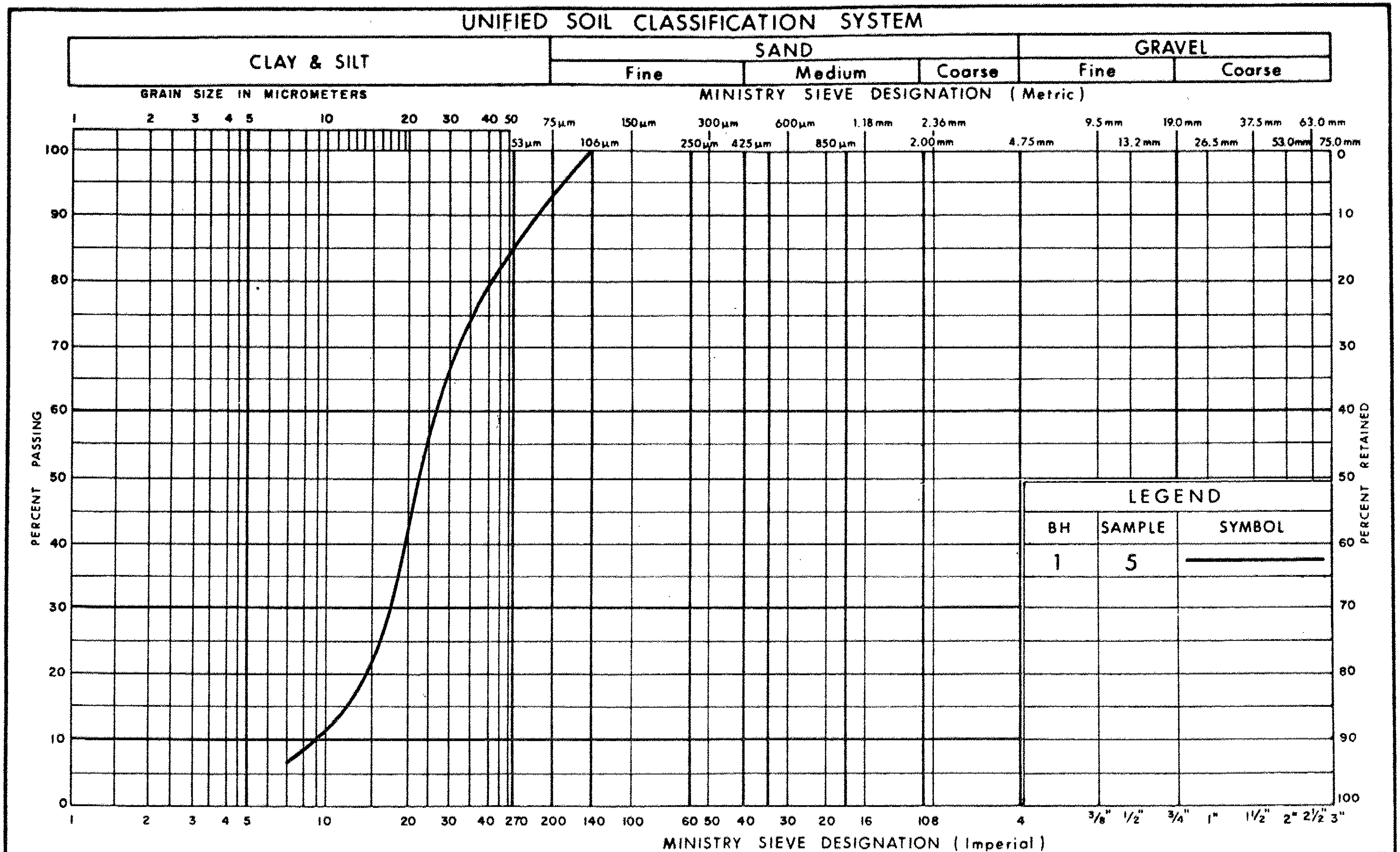
PLASTICITY CHART

CLAY

FIG No 1

W P 145-74-05

Mud Creek



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GRAIN SIZE DISTRIBUTION

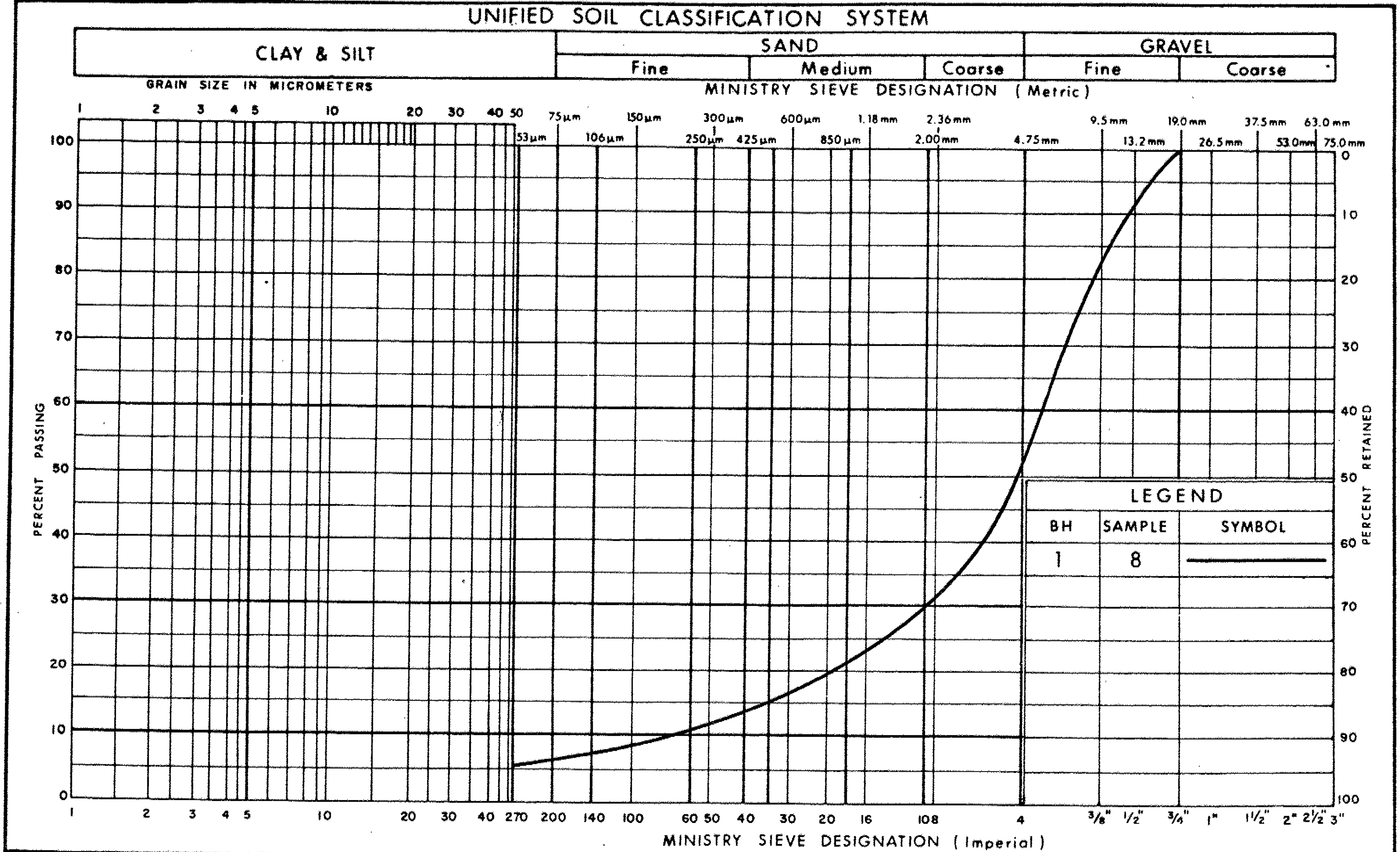
SILT

FIG No 2

W P 145 - 74 - 05

Mud Creek

78 12 M



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GRAIN SIZE DISTRIBUTION
SAND AND GRAVEL (Glacial Till)

FIG No 3

W P 145-74-05

Mud Creek

ENGINEERING MATERIALS OFFICE
PAVEMENT & FOUNDATION DESIGN SECTION

WP 145-74-04 DIST 9

HWY 16N STR SITE 3-356

Hwy. 16N Steven Creek Crossing

CONT 83-15

DISTRIBUTION

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T.J. Kovich (Cover Only)

Files

GEOCRES 31G-190 DATE JAN 19 1983

FOUNDATION INVESTIGATION REPORT (ADDENDUM)

For

W.P. 145-74-04; Site 3-356

Hwy. 16N Steven Creek Crossing

District 9, Ottawa

INTRODUCTION:

This addendum summarizes the results of a foundation investigation required for a proposed structure at the above mentioned site.

Additional fieldwork was undertaken on 82 12 07 (BH 101). Data from previous fieldwork (BH 1 to 4) carried out at the above mentioned location between 81 11 23-24 has been incorporated into this addendum. Total fieldwork consisted of 3 sampled boreholes and 2 dynamic cone penetration tests.

This addendum provides additional data and recommendations required for the design of the proposed structure. However, information contained in the original foundation investigation and design report (dated 81 12 24) remains valid.

Subsurface Conditions

Refer to the appended Record of Borehole Sheets and updated Drawing No. 1457404-A.

Extending from the ground surface downwards, subsurface conditions consist of a surficial organic layer of 0.9 to 1.4 m in thickness, followed by a 4.1 to 4.9 m stratum of highly plastic, sensitive clay. Underlying the clay is a 5 to 6 m thick layer of silt, which contains clay seams. Below the silt stratum is a very dense sand and gravel glacial till of undetermined thickness. Auger refusal on what is probably boulders in the glacial till stratum was encountered at about 14.1 m depth (elevation 72.1 m) at the south abutment location and at approximately 13 m in depth (elevation 72.8 m) at the north abutment location.

Surficial Soils

In the vicinity of the proposed crossing, particularly along the banks of the creek is a plastic organic silt with some fine sand and clay. Natural moisture content determinations were over 82 per cent.

The maximum thickness of this material was 1.5 m at the north abutment location. A Standard Penetration Test 'N' value of 1 blow/0.3 m was determined at one location, indicating the consistency of the deposit may be described as soft. In addition, approximately 200 mm of topsoil was revealed along the higher flat lying ground on either side of the creek banks.

Clay

Underlying the topsoil and organic silts is a 0.9 to 1.4 m thick clay deposit which is identified as Leda Clay. In the area of the southern approach, the upper 2 to 2.5 m of the clay has been dessicated and exibits a natural moisture content in the range of 36 to 46 per cent with mottled gray-brown colour. Below the dessicated zone, the colour of the clay changes to a uniform gray colour, and becomes sensitive to remoulding.

Physical properties of this material as determined from field and laboratory tests are summarized below

			Range	Average
Natural Moisture Content	(w)	%	56-85	72
Liquid Limit	(w_L)	%	52-80	62
Plastic Limit	(w_p)	%	24-31	29
Plasticity Index	(I_p)	%	23-49	33
Shear Strength	(kPa)			
- Field Vane			22-39	27

A plot of plasticity index vs liquid limit (Figure 2) indicates this material is a clay of high plasticity.

The consistency of the clay deposit based on insitu field vanes and laboratory testing indicates the clay can be described as being soft to firm.

Silt

Underlying the clay deposit is an approximate 6 m thick silt stratum. The deposit contains zones of silty clay and distinct 15 mm seams or bands of clay. However, with depth, the frequency of clay seams or bands deminish and the silt becomes much less plastic. Natural moisture contents of the silt was measured to be 32 per cent, whereas the small clay seams were found to be 71 per cent.

Based on 'N' values of 1 to 2 blows/0.3 m, the relative density of the silt can be described as being very loose, whereas its consistency is described as soft to firm in the more plastic zones.

Sand and Gravel (Glacial Till)

Underlying the silt stratum is an undetermined thickness of sand and gravel glacial till. A typical grain size distribution curve appears in Figure 4. The till is comprised mainly of a mixture of sand and gravel, trace of silt and clay.

Standard Penetration Test 'N' values in the till deposit ranged from 5 to 100 blows/0.3 m. Two dynamic cone penetration tests met refusal within this deposit. On the basis of these tests, the overall denseness of the till deposit can be assessed as being dense to very dense. The upper 0.3 to 1 m or so of the deposit possibly have been influenced by contamination with the overlying silt as well as by the presence of artesian flow conditions, contributing to the relatively loose to compact denseness in the upper portion of the glacial till.

Auger refusal on what is probably boulders in the glacial till stratum was encountered at about 14.1 m depth (elevation 77.1 m) at the south abutment location and at approximately 13.0 m depth (elevation 72.8 m) at the north abutment location.

Groundwater Conditions

The groundwater at the north and south abutments at the time of the additional fieldwork was at about creek level (approximate elev. 86±).

Within the underlying glacial till, artesian flow was encountered on both sides of Steven Creek. Estimated pressure heads were 3 m and 2 m above the prevailing creek level at the north and south approaches respectively.

DISCUSSION AND RECOMMENDATIONS

It is proposed to construct a 43 m x 13.5 m single span structure over Steven Creek on the new Hwy. 16N alignment.

Abutment foundations may be supported on steel H-piles equipped with reinforced tips and driven in accordance with MTC Standards SS103-10, 11. The following design values are recommended:

Pile Type	Factored Capacity at U.L.S.	Capacity at S.L.S. Type II
310HP110	1600 kN	1150 kN
310HP 79	1150 kN	820 kN

For estimation purposes, it may be assumed that the recommended pile capacities will be achieved at or below elevation 72.0 m.

Some means of dissipating the artesian flow and preventing piping and erosion is required to the underside of the abutment pile caps. The most practical solution is the provision of a granular filter drainage blanket below the pile cap. The blanket thickness should be approximately 450 mm. If this drainage blanket is situated below high water level, protection with rip rap should be provided to guard against erosion by flooding.

Earth pressures against the abutment walls may be computed as per Subsection 6.6.1.2.2 of the O.H.B.D.C. Manual.

For frost protection, all pile caps should be provided with a minimum of 2 metres of earth cover.

All topsoil and surficial organic material should be removed and the site properly graded prior to the placement of the approach fills.

Long term settlements at the approach fill locations up to 175 mm have been predicted. If possible, construction of the approach fills well in advance of the structure foundations should be considered. Alternatively, paving operations should be delayed for as long as possible so that additional material can be added to bring the approach fills to the required profile grade.

Reference should be made to the original foundation investigation and design report for this W.P. for additional discussion and recommendations.

MISCELLANEOUS

The fieldwork for the addendum portion of the project was carried out under the supervision of Mr. K. Chak, Trainee Engineer, utilizing equipment owned and operated by F. E. Johnston Drilling, Ottawa. The report was written by Mr. Chak and reviewed by Mr. M. Devata, Senior Foundations Engineer.

K. D. Chak
Trainee Engineer

M. Devata, P. Eng.
Senior Foundations Engineer

A P P E N D I X

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 1 — METRIC

W P 145-74-04 — LOCATION STA 19+006.0 O/S 9.0m Rt of ℄ Hwy 16N — ORIGINATED BY CM

DIST 9 HWY 16 N — BOREHOLE TYPE Hollow Stem Auger — COMPILED BY BD

DATUM Geodetic — DATE 1981 11 23 — CHECKED BY CM

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
85.8	Ground Surface					
0.0 – 84.4 / 1.4	TOPSOIL and ORGANIC SILT with Clay and Sand Soft	1	SS	1		
1.4 – 80.3 / 5.5	CLAY, Sensitive Grey Soft to Firm with trace of black Organic inclusions	2	TW	PM	15.2	
		3	TW	PM	15.1 / 15.4	e_o = 2.023, C_c = 1.15, P_o = 18 kPa
		4	TW	PM	15.6	
5.5 – 74.5 / 11.3	SILT, Very Loose (or Soft where Cohesive) with periodic seams of Very Stiff Clay decreasing in frequency with depth	5	TW	PM		
		6	TW	PH	17.2	
		7	TW	PH	16.6	
		8	SS	2		
		9	TW	PM		
11.3 – 72.8 / 13.0	GRAVELLY SAND (Glacial Till) Compact to Very Dense	10	SS	15		
		11	SS	74		40 55 (5)
13.0	END OF BOREHOLE Refusal to Augering					

Ground water conditions: Artesian Head Approx. 3 m Above Ground; Artesian Condition Encountered (at elev. approx. 74.5).

Plot annotations (water content / shear strength): 82% (sample 1), 85% (sample 3); sensitivity values +9, +4, +6, +6, +5.

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (10 20 30 40 50) — O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60.

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 ⌖ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 2

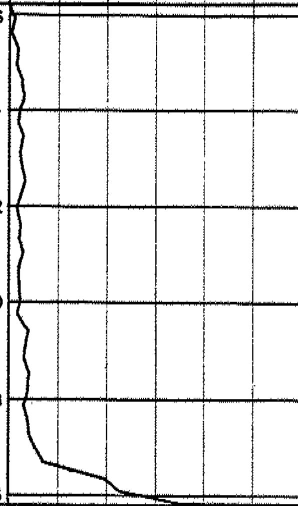
METRIC

W P 145-74-04 LOCATION STA. 19+024.0 O/s 6.0 m Lt. of ℄ Hwy 16 N ORIGINATED BY CM

DIST 9 HWY 16 N BOREHOLE TYPE Cone Penetration Test COMPILED BY BD

DATUM Geodetic DATE 1981 11 23 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L / WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
86.2	Ground Surface										
0.0	Probably CLAY						86, 84, 82, 80				
79.7 / 6.5	Probably SILT						78, 76				
75.8 / 10.4	END OF CONE TEST										



$+^3$, $\times^5$: Numbers refer to Sensitivity

15 ⊕ 5 (20 / 10) (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

OFFICE REPORT ON SOIL EXPLORATION

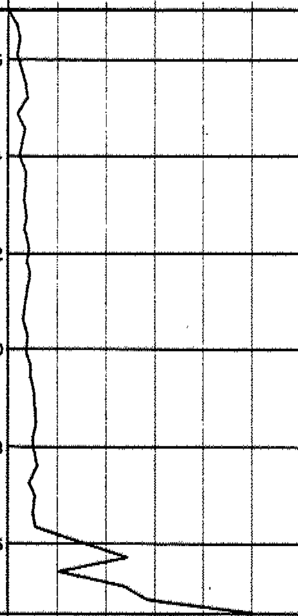
RECORD OF BOREHOLE No 3 — METRIC

W P 145-74-04 — LOCATION STA. 18+973.0 O/s 5.0m Rt ℄ Hwy 16N — ORIGINATED BY CM

DIST 9 HWY 16 N — BOREHOLE TYPE Cone Penetration Test — COMPILED BY BD

DATUM Geodetic — DATE 1981 11 23 — CHECKED BY CM

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
87.0	Ground Surface										
0.0	Probably Desiccated						86				
84.8 / 2.2	Probably CLAY						84, 82				
80.0 / 7.0	Probably SILT						80, 78, 76				
74.5 / 12.5	END OF CONE TEST										



$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 4

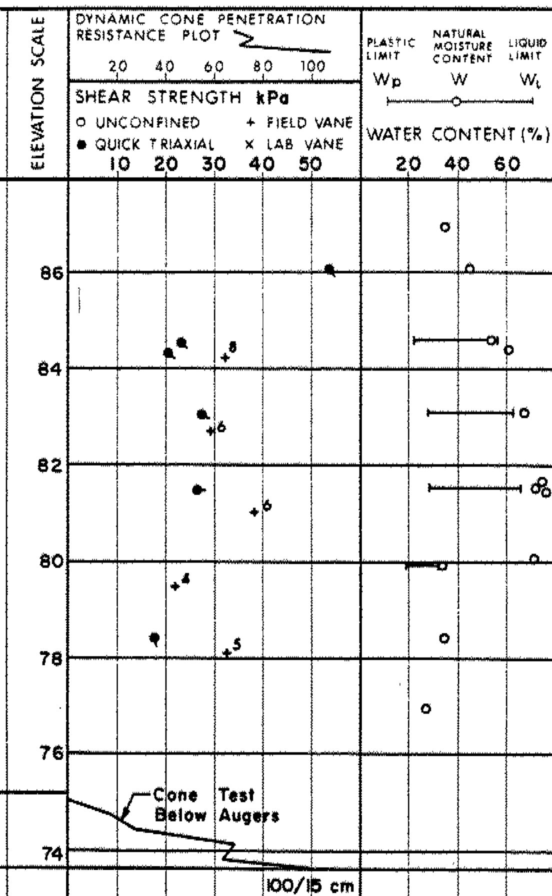
METRIC

W P 145-74-04 LOCATION STA. 18+965.0 O/S 5.0 m Lt of ℄ Hwy 16 N ORIGINATED BY CM

DIST 9 HWY 16 N BOREHOLE TYPE Hollow Stem Auger and Cone Test COMPILED BY BD

DATUM Geodetic DATE 1981 11 24 CHECKED BY CM

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
87.9	Ground Surface						
0.0 / 87.3	TOPSOIL						
0.6	CLAY, Desiccated Stiff Mottled Brown	1	SS	7	▽		
		2	TW	PH		16.8	
84.8 / 3.1	CLAY, Sensitive Grey Soft to Firm with trace of black Organic inclusions	3	TW	PM		16.8	
		4	TW	PM		16.4	
		5	TW	PM		15.5	
80.5 / 7.4	SILT, Very Loose (or Soft where Cohesive) with periodic seams of Very Stiff Clay becoming less frequent with depth	6	TW	PM			
		7	TW	PM		19.1	
		8	SS	1			
75.3		9	SS	6			
12.6	END OF BOREHOLE						
74.4	Probably SILT						
13.5 / 73.7	Probably SAND and GRAV. (Till)						
14.2	END OF CONE TEST						



OFFICE REPORT ON SOIL EXPLORATION

$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 ✧ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 101

METRIC

W P 145-74-04 LOCATION Sta. 18 + 977.0; O/S 6.0 m Lt. ₵ Hwy. 16N ORIGINATED BY KC
DIST 9 HWY 16N BOREHOLE TYPE Hollow Stem Auger COMPILED BY KC
DATUM Geodetic DATE 82 12 07 CHECKED BY [illegible]

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	ELEVATION SCALE
86.2	Ground Surface				
0.0	Topsoil & Organic Silt Soft				86
85.3 / 0.9	Clay, Sensitive Grey Soft to Firm with Trace of Black Organic Inclusions	1	SS	3	
		2	TW	PH	84
					82
		3	TW	PM	
80.4 / 5.8	Silt, Very Loose (or Soft to Firm where Cohesive) with periodic seams of Very Stiff Clay decreasing in frequency with depth				80
		4	TW	PH	78
75.8 / 10.4	Sand and Gravel (Glacial Till) Compact to Very Dense				76
		5	SS	5	
		6	SS	28	74
		7	SS	100/8 cm	
72.1 / 14.1	End of Borehole Refusal to Auger Probable Boulders				

SOIL PROFILE | STRAT PLOT | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
10 20 30 40 50

PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ

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

Artesian Head Approx. 2 m Above Ground

Artesian Encountered

12

8

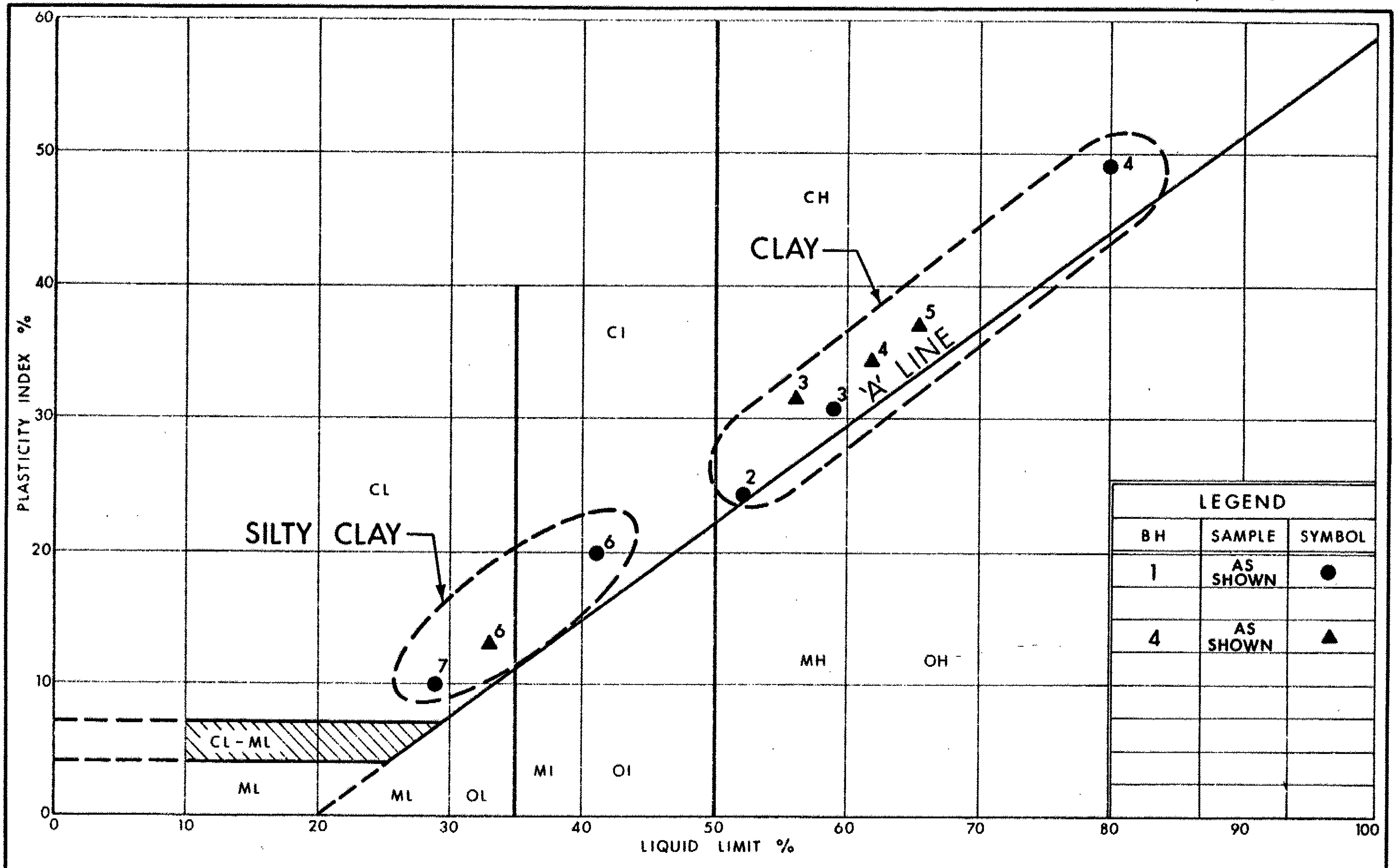
8

$+^3$, $\times^5$: Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{+}} 5$ (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Oct 75, FF-5-21



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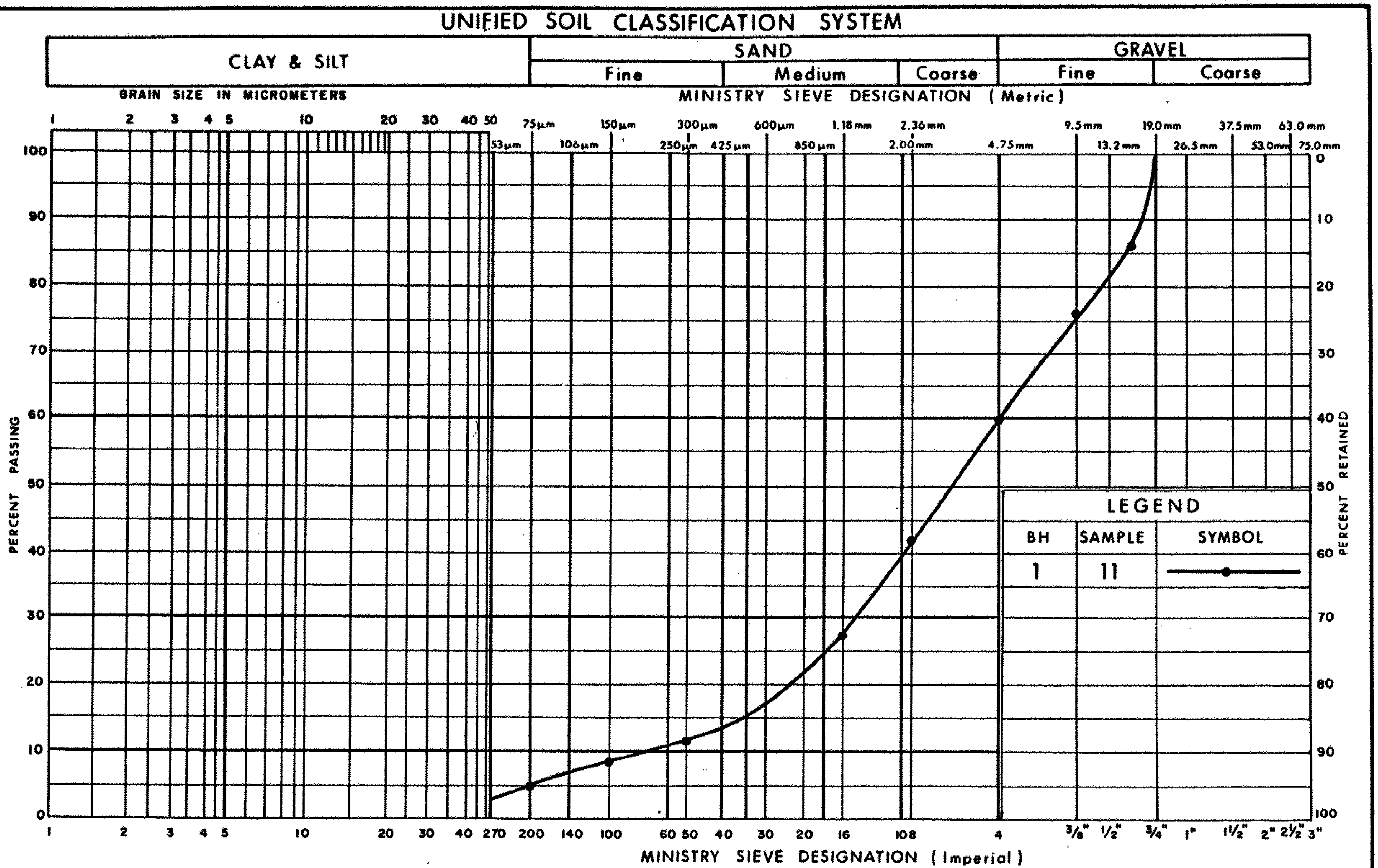
Ontario

PLASTICITY CHART

FIG No 2
WP 145-74-04
Steven Creek

78 12 M



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GRAIN SIZE DISTRIBUTION

SAND AND GRAVEL (Glacial Till)

FIG No 4

W P 145-74-04

STEVEN CREEK

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE

e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO OR-MT-308M 6-78

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

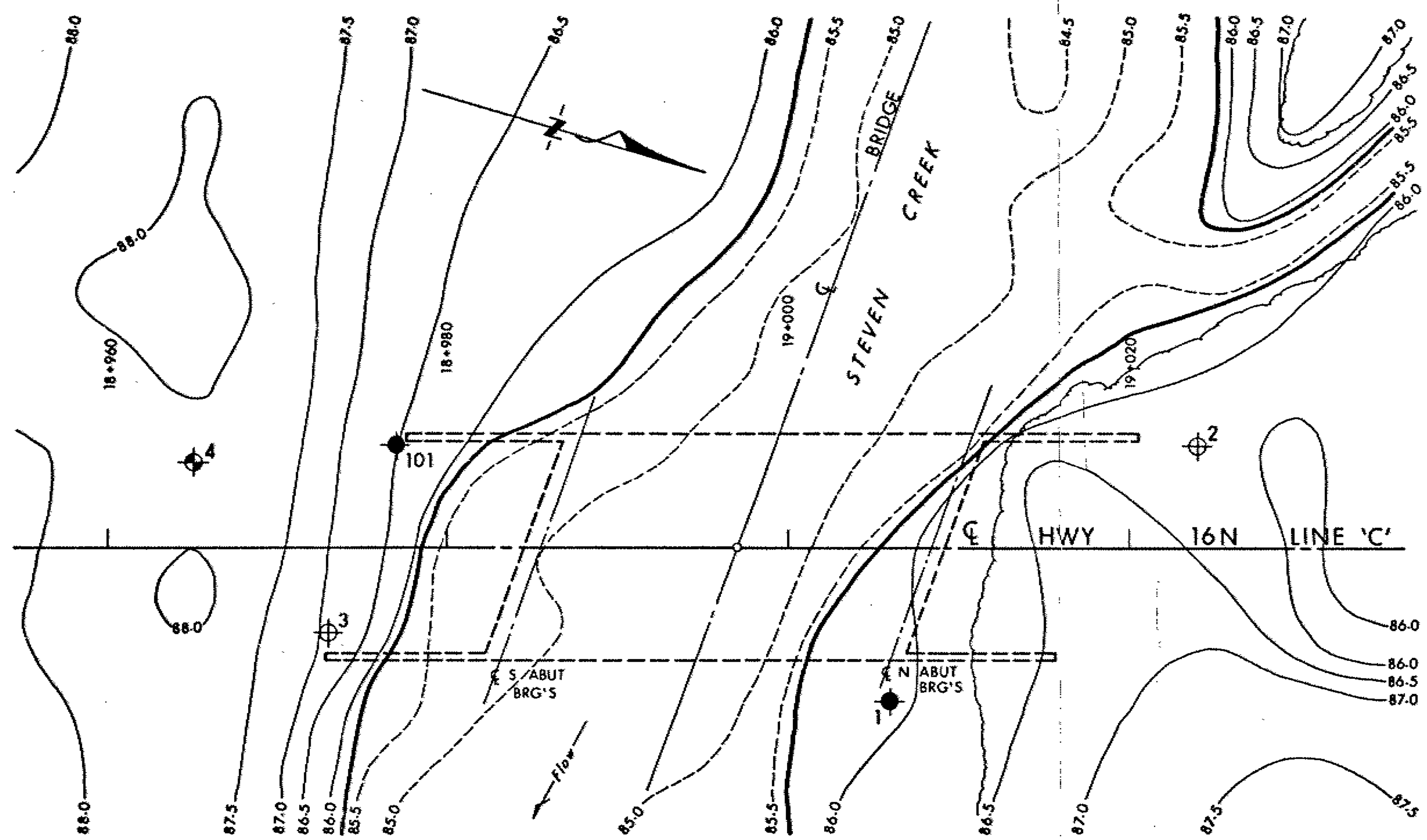
CONT No
WP No 145-74-04

STEVEN CREEK

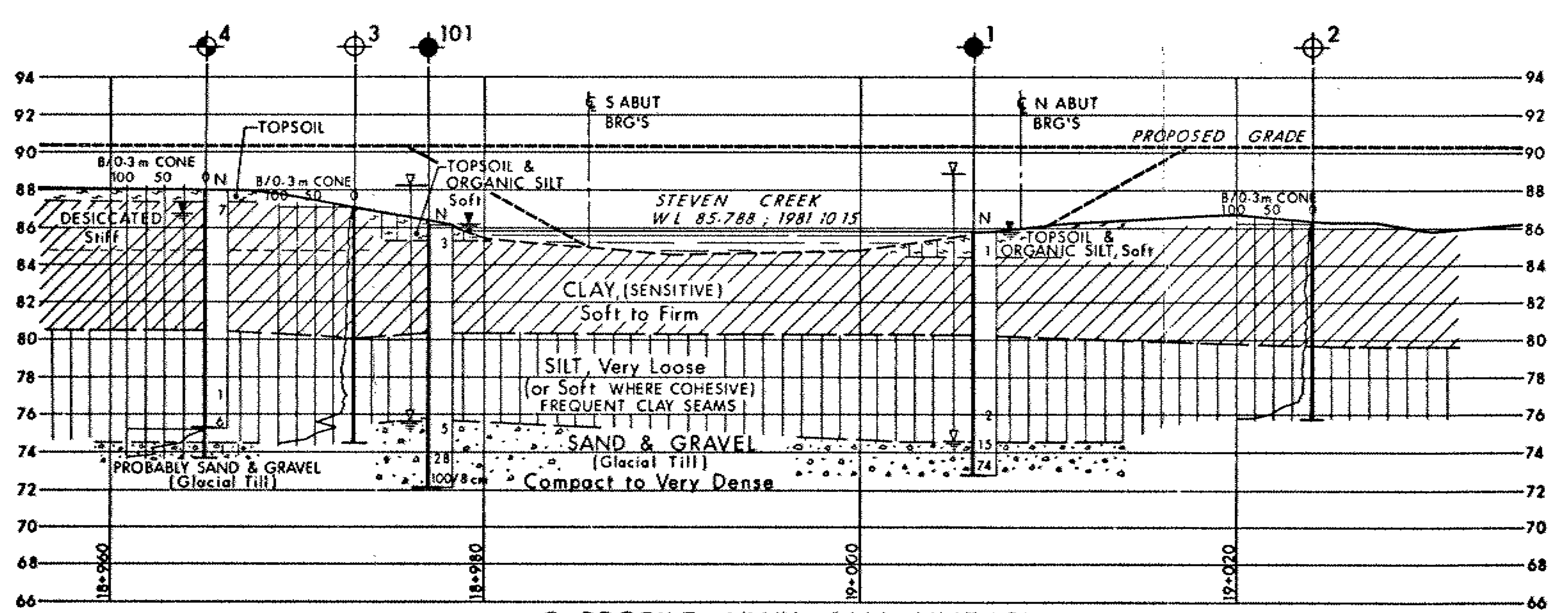
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

Warnock Hersey Professional Services Ltd.



PLAN
SCALE
4m 2 0 4m



℄ PROFILE HWY 16N LINE 'C'
SCALE
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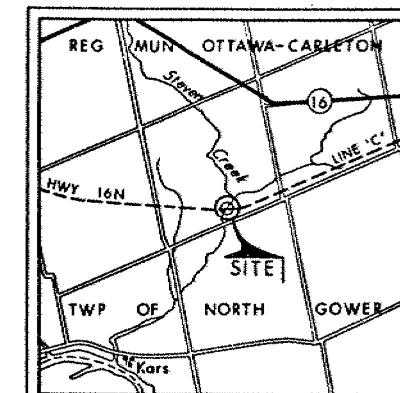

KEY PLAN
SCALE
1km 0 1 2 3km

LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation 1981 11
  WL for Borehole # 101 ; 82 12 07
- Artesian Head
  ARTESIAN CONDITIONS
- Artesian Encountered

No	ELEVATION	STATION	OFFSET
1	85.8	19+006.0	9.0 m Rt
2	86.2	19+024.0	6.0 m Lt
3	87.0	18+973.0	5.0 m Rt
4	87.9	18+965.0	5.0 m Lt
101	86.2	18+977.0	6.0 m Lt

NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS

DATE	BY	DESCRIPTION
82 12 29	GP	BOREHOLE 101 ADDED ON PLAN & PROFILE

Geocres No 31G-190

REF No E-7002-1; 1981 11

HWY No 16N			DIST 9
SUBM'D C M	CHECKED	DATE 1981 12 16	SITE 3-356
DRAWN J T	CHECKED	APPROVED	DWG 1457404-A

memorandum

Ontario

To: Mr. T. C. Kingsland,
Head, Structural Section,
Eastern Region

Date: 82 01 08

CONT 83-15

From: Pavement & Foundation Design Section,
Room 315, Central Building

Re: Foundation Investigation and Design Report by Warnock Hersey Professional Services Ltd., W. P. 145-74-04, Site 3-356, Highway 16N - Steven Creek Crossing, District 9, Ottawa

The M. T. C. retained the services of geotechnical consultants, Warnock Hersey Professional Services Ltd., for the above-mentioned project. The final foundation report was received by this office and subsequently reviewed for technical content. Our comments are as follows:

1. The field investigation was carried out with the assumption that a single-span structure was considered at this site and consequently the investigation was carried out to this concept. However, at a later stage prior to the submission of this report it was understood that a 3 span structure may be considered at this site. As a result of this the field investigation did not fully cover the proposed concept. If a 3 span structure is the final alternative, a further field investigation will be necessary. This Section will initiate the necessary additional investigation.

2. For preliminary design purposes if a three span structure is contemplated the perched abutments can be supported on end bearing Steel 'H' piles driven through the fill material into the underlying compact to very dense glacial till deposit. In such a case rock or bouldery material should not be used for the approach fills in the area where piles are to be driven. The fill material should be restricted to material with a maximum gradation of 75 mm. The pile capacities are as follows:

Factored Capacity at U. L. S.	1,500 kN
Capacity at S. L. S. Type II	1,100 kN

3. Some means of dissipating the artesian flow and preventing piping and erosion is required to the underside of the pier and abutment foundation pile caps. The recommendation pertaining to this are adequately covered on page 14 of the Foundation Report.

JAN 11 1982

7540-1218

4. Long term settlements at the approach fill locations up to 175mm are predicted. If scheduling permits, consideration should be given to construct the approach fills well in advance of the structure foundations. Alternatively paving operations should be delayed to as long a period as possible so that additional material can be added to bring the approach fill to the required profile grade.

We feel that the above comments together with the data contained in the foundation report will be adequate for your design requirements. Should you need further clarification or any additional data feel free to contact us.

M. Devata

M. Devata,
Senior Foundations Engineer

MD/bd

T.C. Kingsland (2)
W.E. Blum
J.W. Reid
R.W. Oddson (2)
K.G. Bassi
B.J. Giroux
R. Hore

L. Saulnier (cover only)
J. Anderson (cover only)
T.J. Kovich (cover only)

Files

FOUNDATION INVESTIGATION REPORT

for

W.P. 145-74-04 Str. Site:3-356

HIGHWAY 16N-STEVEN CREEK CROSSING

District 9 (Ottawa) Eastern Region

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230-2778

1981 12 24

GEOCRES № 31G-190

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FOUNDATION INVESTIGATION REPORT
for
W.P. 145-74-04
Structure Site 3-356
Proposed Crossing at Steven Creek and Hwy. 16N
District of (Ottawa), Eastern Region

1. INTRODUCTION

Warnock Hersey Professional Services Limited have been retained by the Ontario Ministry of Transportation and Communications, under Agreement No. 4242-9081-108, to provide geotechnical engineering services in connection with the above project. The terms of reference were to carry out a site investigation of sufficient scope to provide recommendations for the design and construction of a new bridge at this site.

The request was received on 1981 11 12. A letter dated 1981 11 27 was submitted upon completion of the field work, with some preliminary recommendations. This report describes the field work and soil conditions in greater detail. Recommendations are given for the design and construction of foundations and embankments of the proposed structure and its approaches.

2. DESCRIPTION OF SITE AND GEOLOGY

The site is located in North Gower Township, some 35 kilometers south of Ottawa. Specifically, the proposed structure site is located 1.3 km north of Roger Steven Road (Cty Rd. 4), some 300 m west of 2nd Line Rd. At this location, Steven Creek is approximately 18 m wide and 3 m deep. The flow is in an easterly direction towards the Rideau River.

The area is designated physiographically as the North Gower Drumlin Field. Air photos of the area and ground observations show the presence of buried drumlins in the vicinity. These drumlins are oriented more or less north-south. A mantle of marine soil is draped over these drumlins and other similar glacial inorganic land forms.

2. DESCRIPTION OF SITE AND GEOLOGY (Cont'd)

The marine soil originated with the inundation of the area by the Champlain Sea. Therefore, silts and Leda clays are common in this area.

Due to the undulating nature of the topography, several springs have developed in the area. The spring waters originate in the higher ground where the mantle of marine soil is thin. The higher ground is generally a glacial landform. Springs are frequently found in the low ground between two relatively prominent glacial landforms, particularly where erosion has removed much of the cohesive marine soil mantle. Elsewhere, artesian groundwater conditions are generally encountered at depth.

At this site, the south approach of the proposed crossing consists of a cleared farmers field. The ground sits about 2 to 3 meters higher than the north approach, where the land is bush covered. A drainage tributary stream flows in from the north to join Steven Creek near the north approach area. Within the bush covered area, there are ox-bow shaped remnants of former stream tributary meanders. Both the north and south banks of the stream support swampy vegetation such as reed grass and sedges. There are no signs of slope instability, past or present, along the higher south bank of the creek near the proposed crossing.

3. FIELD AND LABORATORY WORK

The field work consisted of the drilling of two sampled boreholes, one on each side of the creek, supplemented by two additional dynamic cone penetration tests. The investigation was carried out by means of a muskeg vehicle mounted continuous flight auger machine, equipped with hollow stem augers (item 5.2.i of MTC Supply Contract No. S-80-3298). The location of the bore and cone holes is shown on drawing 1457404-A in the Appendix. All bore and cone holes were terminated within a very competent glacial till deposit.

3. FIELD AND LABORATORY WORK (Cont'd)

Strong artesian flow was encountered at the north approach borehole (BH1) once the till was penetrated. It was thought prudent to terminate the drilling within the till in order to control the groundwater flow.

Standard Penetration Resistance tests were carried out in the hard or very dense materials as well as in the non-cohesive zones of the stratigraphy. Elsewhere, 51 mm thin-walled steel tube samplers were pushed in manually or hydraulically to recover relatively undisturbed samples of the cohesive soil strata. Some silt samples recovered by this procedure began to "flow" and were transferred into sealed glass jars. Where feasible, laboratory size vane shear tests were carried out at the tip of each tube in the field immediately upon recovery of the sample, prior to capping and sealing of the tubes.

After recovery of each thin-wall tube sample, standard M.T.C. "A" size vane tests were carried out some 457 mm below the last sample. The undrained field vane shear strengths were noted. After initial failure, the vane was rotated six times quickly to remould the soil, care being taken to ensure the vane and rod asssembly did not sink into the soil when this was being done. After a 60 second set-up period, the remoulded shear strength was measured. The ratio of the undisturbed to remoulded undrained shear strength was recorded as the sensitivity of the soil, to the nearest whole number.

Upon completion of each borehole, water level readings were taken in the uncased holes. At the location of Borehole 1, the artesian flow was estimated by visual means and the head was estimated by adding augers to serve as a standpipe. Due to leakage between the auger joints, the artesian head could not be accurately measured; hence, the reported value is a best estimate.

3. FIELD AND LABORATORY WORK (Cont'd)

All soil samples were protected from freezing during the investigation. They were transported each evening to a warm room and stored there for ultimate transportation by car to our laboratories in Toronto. The samples were placed inside the car both for warmth and lessening of disturbance during transportation.

In the laboratory, the soil samples were visually examined and assigned for further testing. The laboratory testing consisted of:

Natural Moisture Contents
Atterberg Limits (on Undried samples)
Grain Size Distributions
Unconsolidated Undrained Triaxial Tests
Consolidation Test

The results of these tests are plotted on individual Records of Boreholes as well as on the Figures in the Appendix.

The samples remaining after testing were transferred into sealed glass jars and will be stored for future reference. All thin-walled tube samples were extruded; unused portions were dissected longitudinally for close examination of the sedimentation characteristics. The dissected specimens are stored in sealed glass containers.

4. SUBSOIL CONDITIONS

4.1 General

The soil conditions across the site are fairly consistent. Briefly, the soil stratigraphy consists of a surficial organic layer followed by a highly plastic, sensitive clay, overlying a silt deposit. The silt stratum, which contains clay seams and silty clay zones, is underlain by a very dense sand and gravel glacial till of undetermined thickness. The details follow:

4.2 Surficial Organics

Along the higher flat-lying land on either side of the stream banks, the topsoil is about 200 mm in thickness. Along the shores of the river, particularly the north shore line in the vicinity of the proposed crossing, the upper 1.5 metres consists of a plastic organic silt with some fine sand and clay. This material was fairly compressible but not spongy. It's moisture content was over 82 percent. Similar material likely exists along the flatter portions of the south shore banks of the creek. An N value of 1 blow/0.3 m was recorded at one location in this deposit. It's consistency is described as soft.

4.3 Clay

Immediately below the topsoil on the flatter ground, or organic silts along the shores, there is present a clay deposit which is identified as Leda clay.

The thickness of this deposit varies depending on surface relief. At the south approach, it is approximately 7 metres thick whereas at the north approach it is only about 4 metres thick. In either case, however, the deposit extends down to about elevation 80.5.

The engineering properties of this deposit are summarized on Figure 1 along with the properties of the underlying strata. The disinguishing characteristics are enumerated in Table 1 for the south and north approaches:

Table 1 Physical and Engineering Properties of "Undesiccated" Clay at Steven Creek

	North Approach		South Approach	
	Range	Mean	Range	Mean
Moisture Content (%)	74-85	78	56-75	65
Liquid Limit (%)	52-80	64	56-66	60
Plastic Limit (%)	28-31	30	24-29	27
Plasticity Index (%)	23-49	34	32-37	33
Liquidity Index	0.9-2.0	1.6	1.0-1.2	1.1
Unit Weight (kN/m^3)	15.2-15.6	15.3	15.5-16.8	16.2
Undrained Shear Strength (kPa)				
Field Vane	22-29	25	30-39	33
UU Triaxial	20-29	25	21-28	25
Lab Vanes- All values were lower than field or triaxial values				
Sensitivity	4-9	6	6-8	7

Preconsolidation, P_c, (kPa)	100-105
C_c	1.15
C_r	0.07
C_v (mm^2/sec)	0.5

The south approach ground surface sits some 2 to 2.5 metres higher than the north approach ground surface. Hence, at the south approach, the upper 2.5 metres or so of the clay has been desiccated and exhibits a natural moisture content in the range of 36 to 46 percent, with an average of 42. In this desiccated zone, the undrained shear strength measured by the UU Triaxial test ranged from 68 to 54 (kPa), decreasing rapidly with depth. Significant strength differences were found within each thin-walled tube near the transition zone from desiccated to undesiccated.

4.3 Clay (Cont'd)

The soil in the desiccated zone has a mottled grey-brown colouring. Below the desiccated zone, the soil exhibits a fairly uniform grey colour with black spots and streaks representing fossil remnants now turned organic due to lack of oxygen in a reducing environment.

An examination of the soil properties in Table 1 shows some interesting differences. The mean moisture content of the clay below the desiccated zone is significantly lower than that where no desiccated crust is present. This difference is reflected also in the other index properties, including the unit weight.

The Atterberg limits of the clay are plotted on Figure 2. It can be seen that the plasticity of the soil increases with depth, each subsequent sample plotting above the last sample along the A-line.

There was a significant difference observed in the field vane shear strengths between the south and north approach clays. Where the desiccated zone is present, the field vanes indicate a substantially higher undrained shear strength than where the desiccated zone is absent. A similar difference was not noticed in the UU triaxial tests. The laboratory vane tests are not reported since the values measured in the field on the recovered samples were well below those measured by the field vane and in the triaxial test. This would indicate a significant influence of scale on the measured shear strengths. The laboratory vane used was a 6-bladed, commercial, pocket sized torvane.

It is reasonable to assume from the data given above that the removal of the desiccated zone at the north approach has affected to a considerable extent the physical properties of the clay. The removal of overburden (desiccated zone) by erosion has resulted in vertical stress relief followed by moisture intake and reduced shear strengths.

A sample of the clay from the borehole at the north approach was tested to determine its preconsolidation value. The result is shown on Figure 3. The e-log p curve in Figure 3 is typical of Leda clay soils. The preconsolidation is estimated at 100 to 105

4.3 Clay (Cont'd)

kPa. Part of this high value is no doubt attributable to erosion of 2.5 to 3 metres of overburden. Assuming the overburden weighs 17 kN/m^3, the stress relief is in the order of 50 kPa. Therefore, only 50 kPa (100-50) represents preconsolidation due to either chemical bonding or aging of the clay. These relatively high values for this soil are confirmed by extensive testing carried out by the National Research Council of Canada for the instrumentation of the Rideau River Bridge at Kars, some 2.5 kilometers distant from this site. The NRC consolidation testing of Leda clay from Kars gave preconsolidation (p_c) values ranging from about 90 kPa (minimum estimate) to as high as 170 kPa (the most probable value being around 130 kPa). Therefore, the tested value of around 100 kPa fits within the NRC range of tested values.

4.4 Silt

At elevation 80.5, the clay deposit transitions into a silt deposit which extends to the glacial till stratum. The thickness of the silt deposit is about 6 metres. The deposit is non-homogeneous in that it contains zones of silty clay and distinct seam or bands of clay. In a typical sequence of stratification, the soil consists of a 100 mm layer of silt, a 15 mm layer of brittle hard clay, a 70 to 100 mm layer of slightly plastic silt, then another layer of non-cohesive silt. With depth, the silt layers become thicker and less plastic; the frequency of occurrence of the thin clay bands or seams diminishes. In the last one metre or so of the deposit, some sand and gravel is present.

The silt layers are extremely dilatant. However, the clay seams or bands were found to be brittle and very stiff. The clay is highly plastic and exhibits a cubic fabric structure and fractures conchoidally. It's texture is smooth.

One careful separation of the two soil types was carried out to measure their moisture contents. The moisture content of the silt was measured to be 32 percent whereas that of the immediately adjoining clay seam was found to be 71 percent.

4.4 Silt (Cont'd)

During deposition, the silt has become contaminated with clay and therefore exhibits some plasticity characteristics. The composite effect is a soil mixture having the plasticity characteristics of a silty clay, as shown on Figure 2. Vane tests were taken in this deposit in the belief that it was a plastic soil. However, careful examination of the extruded tube samples reveals the soil to be a silt; therefore, the vane shear strength values may not be an appropriate measure of the strength of the silt.

The physical properties of the silt deposit are summarized in Table 2.

Table 2 Physical Properties of Silt Deposit at Steven Creek

Physical Property	Siltier Portion		Clay Portion	
	Range	Ave.	Range	Ave.
Moisture Content (%)	21-32	26	32-71	45
Liquid Limit (%)	N.P.	-	29-41	35
Plastic Limit (%)	N.P.	-	19-21	20
Plasticity Index (%)	-	-	10-20	15
Undrained Shear Strength (kPa)				
Field Vane	Range: 22-42			
UU Triaxial	Range: 18-35			
Unit Weight (kn/m^3)	Range: 16.6-19.2			

Based on "N" values of 1 to 2 blows/0.3m and the low undrained shear strengths, the relative density of the silt is described as very loose whereas its consistency is described as soft in the more plastic zones.

4.5 Sand and Gravel (Glacial Till)

Below about elevation 75, the silt deposit is underlain by a glacial till consisting essentially of a mixture of sand and gravel with a trace of silt and clay. A grain size distribution curve is shown in Figure 4. The sand and gravel particles are angular. The predominant constituent is a dark grey limestone which indicates the till to be a basal till derived largely from the local bedrock in the area. The material at first glance looks like weathered bedrock. However, there are also present, in the sand-sized grains, orthoclase feldspar, quartz and calcite components indicating the material to be of glacial origin and not a product of in-situ weathering.

N values in this till deposit ranged from 15 to 74 blows/0.3m. Two dynamic cone penetration tests met refusal within this deposit. On the basis of these tests, the relative density of the sand and gravel is considered to be very dense. The upper 0.3 metres or so of the deposit has been influenced by contamination with the overlying silt. Therefore, it exhibits a lower relative density (N value of 15 blows). From the results of other soil investigations in the vicinity, it is believed that bedrock may be located not to far below the termination of the boreholes and cone tests within this deposit.

5. GROUNDWATER CONDITIONS

The groundwater at the north approach is located at about creek level. At the south approach, the groundwater was about 1 metre below the ground surface. Therefore, the phreatic surface is presumed to follow the ground profile. Fluctuations of the groundwater level in the clay are likely to be mimimal when the creek floods. From the dessication observed in the clay deposit at the south approach, the long-term water level is assumed to prevail at a depth of about 2 metres.

Within the underlying glacial till, artesian flow was encountered. The estimated pressure head at the north approach is 3 metres above prevailing creek level. A very minor artesian condition was noted at Borehole 4 at the south approach. It is

5. GROUNDWATER CONDITIONS (Cont'd)

believed that the higher ground on the south shore is characterized probably by sub-artesian conditions.

The farmer-property owner on the east side of the north approach stated that springs are present on his property about halfway between the proposed crossing and 2nd Line Road. He claims such springs are quite common in the area.

6. DISCUSSION AND RECOMMENDATIONS

6.1 General

At the time of authorization to proceed with the field investigation, E-plans of the site were not available. It was understood a single span bridge or culvert was being contemplated. The field investigation was accordingly conducted. However, it is now understood from the E-plan provided that a three span bridge is proposed at this site. The two piers will be within the creek channel. Due to hydraulic considerations, the north abutment has been located in such a position as to not interfere with the flow into the creek from the small tributary stream or ditch entering the site from the north west.

The proposed spans are 8, 14 and 8 metres respectively. The deck width will be 12.5 metres and the bridge will be placed at a 30 degree skew to the creek. The profile grade at the crossing is horizontal and located at about elevation 89.5. Hence, the maximum height of approach fills will be in the order of 3.5 metres.

The soil investigation has revealed the presence of a firm clay overlying loose or soft silt above a very dense glacial till deposit. Locally, the surficial soils are organic and on higher ground, the clay is dessicated in the upper two metres. The clay reveals a fairly significant amount of preconsolidation.

6.2 Foundation Design

The presence of relatively compressible and weak soils at this site preclude the use of conventional spread footing type foundations for the structure. Therefore, a deep foundation is recommended.

Deep foundations must contend with artesian and sub-artesian conditions within the bearing stratum, which is a sand and gravel till. Caissons are not practical. Therefore, piles driven to end bearing on or within the till are recommended.

6.2 Foundation Design (Cont'd)

Due to the artesian condition, two types of piles should be costed out. These are timber and steel H piles. If steel H piles are used, certain additional site modification costs shold be included in the economic analysis for comparative purposes. The details are as follows:

6.2.1 Timber Piles

Timber piles are tapered and form a natural seal as they are driven tip first into the soil. The soils at this site will not prevent the driving of timber piles to the glacial till stratum. However, once the piles reach the sand and gravel till stratum, further penetration will be difficult without damage to the pile heads.

For the proposed heights of fill, the loading on the clay will be below the preconsolidation pressure of the clay. Therefore, settlements of the fills will be negligible. Hence, negative skin friction will not be a factor in the design of the timber pile. Therefore, the factored capacity at ultimate limit state should be based on the pile material and cross-section used.

For 330 mm diameter, red pine or equivalent pressure creosoted piles, driven to or below elevation 74 into the glacial till, the following design values are recommended:

Safe capacity	300 kN
Factor capacity at U.L.S.	790 kN
Capacity at S.L.S. Type II	300 kN

6.2.2 Steel H-Piles

Steel H piles, by virtue of carrying larger axial loads than timber piles, may prove more economical depending on the magnitude of total loads to be carried. Such piles can be driven into the glacial till. Their capacities may be assumed to be proportional to the cross-sectional area. For 310 HP @ 110 steel piles, driven with a minimum energy of 50 kJ per blow to or below tip elevation 73.0, the following design parameters may be used

Safe capacity	1,100 kN
Factor capacity at U.L.S.	1,500 kN
Capacity at S.L.S. Type II	1,100 kN

The H piles may or may not form a natural plug as they are driven through the cohesive overburden. Therefore, the possibility exists of artesian leakage along these piles. Such leakage could result in piping and erosion of the subsoil with consequent loss of ground. Therefore, some means of dissipating the artesian flow pressures and preventing piping and erosion is required. The most practical solution, which has been tried by the MTC with success in the past, is the provision of a granular filter-drainage blanket below the pile cap. After site clearings and all necessary sub-excavations, the granular blanket should be laid to a thickness of at least 450 mm. If this blanket is situated below design high water, it should be protected with rip-rap against erosion by flooding.

6.3 Approach Embankments

The proposed profile grade at elevation 89.5 results in maximum heights of approach fills of in the order of 3.5 metres.

6.3.1 Stability

From the summary plot of engineering properties shown on Figure 1, it can be seen that the average undrained shear strength of the clay is about 25 kPa. Therefore the proposed approach fills will be stable against deep-seated rotational type failure through the clay deposit. Care shold be taken to ensure

6.3.1 Stability (Cont'd)

that the approach fills are built with side slopes no steeper than 2:1 and with material whose compacted in-place density does not exceed $21 kN/m^3$.

The very loose silt deposit occurs at about elevation 80, some 6 metres below the ground surface. Assuming $c'_F = 0$ and $\phi'_F = 18°$ for this material, sliding wedge analyses indicates adequate resistance against ultimate failure through the clay and at the clay-silt interface.

6.3.2 Settlement

The most probable preconsolidation of the clay below the approach fills is about 100 kPa. Assuming a unit weight of fill material of $21 kN/m^3$, the height of fill required to exceed this preconsolidation pressure is about 4.8m. The proposed approach fills are about 3.5 m in height. Therefore, the net stress change on the clay will be below the established most probable preconsolidation pressure of the clay. Hence, settlements are likely to be of a recompression type and minor in magnitude.

At the abutments, where the approach fill heights are a maximum of about 3.5 metres, the estimated total settlement of the clay and silt deposits is in the order of 175 mm over the long term. It is expected that 50 percent of this magnitude of settlement will occur within the first three years after construction. If time is available, consideration should be given to pre-loading of the site. Surcharging is not recommended due to stability considerations.

6.3.3 Earth Pressure

The proposed design calls for closed type abutments. Earth pressures against abutment walls may be computed as per Subsection 6.6.1.2.2 of the O.H.B.D.C.

6.4 Other Design Considerations

All pile caps should be provided with a minimum of 2 metres of earth cover for protection against frost heaving.

The sliding resistance of the base of pile caps, between concrete and the clay, may be assumed to be 4 kPa, since the clay is sensitive and excavation will disturb the surface.

If wing walls are contemplated, the ends of such walls should be supported on end bearing piles, unless they can be adequately cantilevered out from the main abutment. The settlement of the approach fills behind the abutments is expected to be minor and therefore "tilting" of the abutments is not anticipated. However, provision of piling support at the wing wall tips would be a desirable design feature.

6.5 Construction Considerations

6.5.1 General

The recommendations provided in this report are based on data collected at and interpreted from a limited site investigation. If soil, rock or groundwater conditions different from those described or assumed in this report are uncovered during construction, we should be consulted at once to determine and advise if modifications from a geotechnical viewpoint are necessary to the design or construction aspects of the project and, so to ensure its continued economy and safety.

6.5.2 Footing Excavations and Dewatering

Excavations in the clay may be made at temporary 1:1 side slopes. Even though the groundwater table is fairly high, infiltration through the clay is expected to be minimal. Hence, groundwater seepage may be handled by pumping from sumps. Excavations in the dessicated crust on the south approach may experience some seepage of groundwater through fissures in the deposit.

6.5.2 Footing Excavations and Dewatering

Because of the sensitive nature of the clay and the difficulty which might be experienced by workmen in excavations in such clays, it would be advisable to provide for a 150 mm cushion of sand at the bottom of the excavation as a working base. The sump level should be below the sand cushion.

The pier footings will be situated within the stream. The pile caps could be constructed in the dry by building earth cofferdams around these locations. However, care should be taken to ensure the cofferdam does not fail into the excavation, by maintaining a maximum distance between base of excavation and top of cofferdam of 3 metres at an average slope no steeper than 1-1/2:1, for an unsupported excavation. Alternatively, the pile caps may be precast as concrete box caissons with templates to guide the piles for driving, after which the box can be filled up with concrete.

6.5.3 Approach Fills

The shore line contains organic deposits of nominal thickness. These should be excavated from below the plan limits of the approach fills. In Drawing 1457404-A, the south approach shows clay. Since no borings were done at the south shore line, it has not been feasible to show organics at this location on the drawing. However, from the general site characteristics, such organics are suspected at both the north and south shore lines. Therefore, provisions should be made for removal of such organic soils down to the native clay stratum.

Backfilling of subexcavated areas should be with a clean non-cohesive free draining material such as Granular "B" or "C".

The base of the approach fill should be rip-rapped to flood level of the stream, based on its hydrological design criteria.

7. CONCLUSIONS

The site conditions are not feasible for the use of spread footings. Due to artesian conditions at depth, deep foundations should consist of timber piles or steel piles with a granular blanket below the pile cap to dissipate any upward seepage flow along the pile sides. Settlement of fills are expected to be minor due to prevailing preconsolidation of the natural clay. No stability problems are anticipated.

Respectfully submitted,

Warnock Hersey Professional Services Ltd.

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Encl.

A P P E N D I X

Office Record of Boreholes 1 to 4
Figures 1 to 4
Drawing 1457404-A

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3 m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON
W S	WASH SAMPLE
S T	SLOTTED TUBE SAMPLE
B S	BLOCK SAMPLE
C S	CHUNK SAMPLE
T W	THINWALL OPEN
T P	THINWALL PISTON
O S	OSTERBERG SAMPLE
R C	ROCK CORE
P H	T W ADVANCED HYDRAULICALLY
P M	T W ADVANCED MANUALLY
F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_p	%	PLASTIC LIMIT
w_s	%	SHRINKAGE LIMIT
I_p	%	PLASTICITY INDEX $= w_L - w_p$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_p}{I_p}$
I_c	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_p}$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 1

METRIC

W P 145-74-04 LOCATION STA 19+006.0 % 9.0m Rt of ℄ Hwy 16N ORIGINATED BY CM

DIST 9 HWY 16 N BOREHOLE TYPE Hollow Stem Auger COMPILED BY BD

DATUM Geodetic DATE 1981 11 23 CHECKED BY CM

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Artesian Head Approx. 3 m Above Ground					
85.8	Ground Surface					
0.0 – 84.4 / 1.4	TOPSOIL and ORGANIC SILT with Clay and Sand Soft	1	SS	1		82%
1.4 – 80.3 / 5.5	CLAY, Sensitive Grey Soft to Firm with trace of black Organic inclusions	2	TW	PM	15.2	
		3	TW	PM	15.1 / 15.4	85%; e_o = 2.023, C_c = 1.15, P_o = 18 kPa
		4	TW	PM	15.6	
5.5 – 74.5 / 11.3	SILT, Very Loose (or Soft where Cohesive) with periodic seams of Very Stiff Clay decreasing in frequency with depth	5	TW	PM		
		6	TW	PH	17.2	
		7	TW	PH	16.6	
		8	SS	2		
		9	TW	PM		Artesian Condition Encountered
		10	SS	15		
11.3 – 72.8 / 13.0	GRAVELLY SAND (Glacial Till) Compact to Very Dense	11	SS	74		40 55 (5)
13.0	END OF BOREHOLE Refusal to Augering					

Elevation scale: 84, 82, 80, 78, 76, 74

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 10 20 30 40 50

Sensitivity values plotted: +9, +4, +6, +6, +5

PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

+³, ×⁵ : Numbers refer to Sensitivity

15 ⊹ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 2

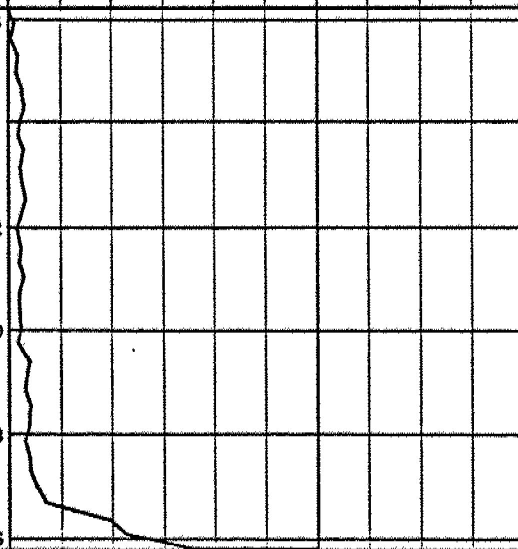
METRIC

W P 145-74-04 LOCATION STA. 19+024.0 o/s 6.0 m Lt. of ℄ Hwy 16 N ORIGINATED BY CM

DIST 9 HWY 16 N BOREHOLE TYPE Cone Penetration Test COMPILED BY BD

DATUM Geodetic DATE 1981 11 23 CHECKED BY CM

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
86.2	Ground Surface										
0.0	Probably CLAY						86, 84, 82, 80				
79.7 / 6.5	Probably SILT						78, 76				
75.8 / 10.4	END OF CONE TEST										



$+^3$, $\times^5$: Numbers refer to Sensitivity

15 -◇- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION – ENGINEERING MATERIALS OFFICE – SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 3

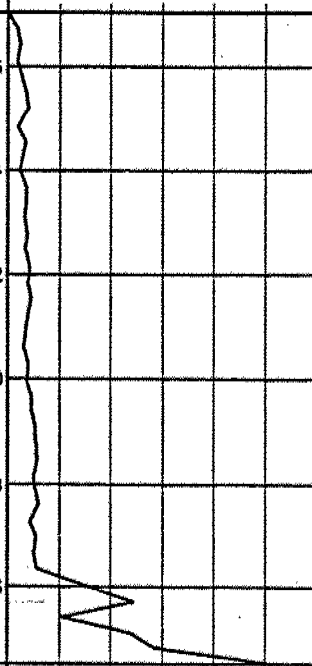
METRIC

W P 145-74-04 LOCATION STA. 18+973.0 O/s 5.0 m Rt ℄ Hwy 16N ORIGINATED BY C M

DIST 9 HWY 16 N BOREHOLE TYPE Cone Penetration Test COMPILED BY B D

DATUM Geodetic DATE 1981 11 23 CHECKED BY C M

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, x LAB VANE	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
87.0	Ground Surface										
0.0	Probably Desiccated						86				
84.8 / 2.2	Probably CLAY						84, 82				
80.0 / 7.0	Probably SILT						80, 78, 76				
74.5 / 12.5	END OF CONE TEST										



$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 4

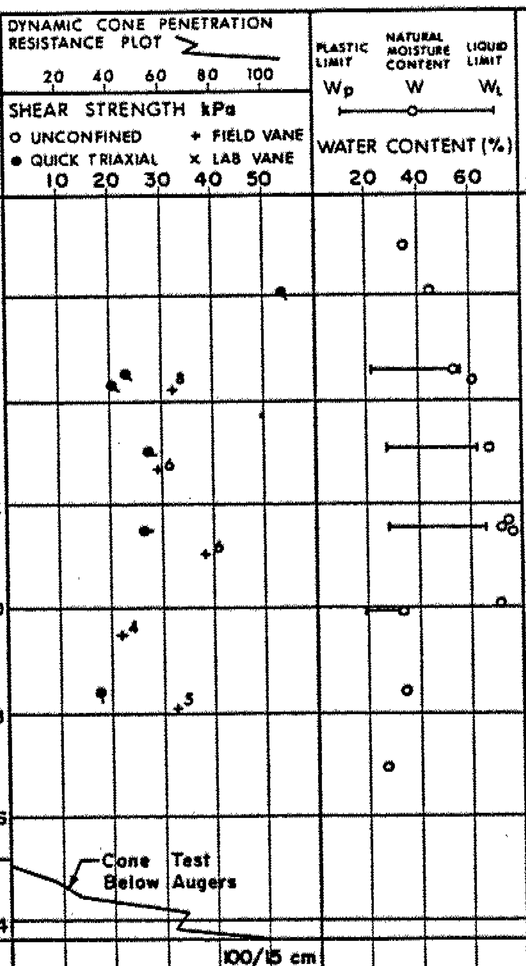
METRIC

W P 145-74-04 LOCATION STA. 18+965.0 O/S 5.0 m Lt of ℄ Hwy 16 N ORIGINATED BY CM

DIST 9 HWY 16 N BOREHOLE TYPE Hollow Stem Auger and Cone Test COMPILED BY BD

DATUM Geodetic DATE 1981 11 24 CHECKED BY CM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³
87.9	Ground Surface				
87.3 / 0.0	TOPSOIL				
0.6	CLAY, Desiccated Stiff Mottled Brown	1	SS	7	
		2	TW	PH	16.8
84.8 / 3.1	CLAY, Sensitive Grey Soft to Firm with trace of black Organic inclusions	3	TW	PM	16.8
		4	TW	PM	16.4
		5	TW	PM	15.5
80.5 / 7.4	SILT, Very Loose (or Soft where Cohesive) with periodic seams of Very Stiff Clay becoming less frequent with depth	6	TW	PM	
		7	TW	PM	19.1
		8	SS	1	
75.3		9	SS	6	
12.6	END OF BOREHOLE				
74.4	Probably SILT				
13.5 / 73.7	Probably SAND and GRAV. (Till)				
14.2	END OF CONE TEST				



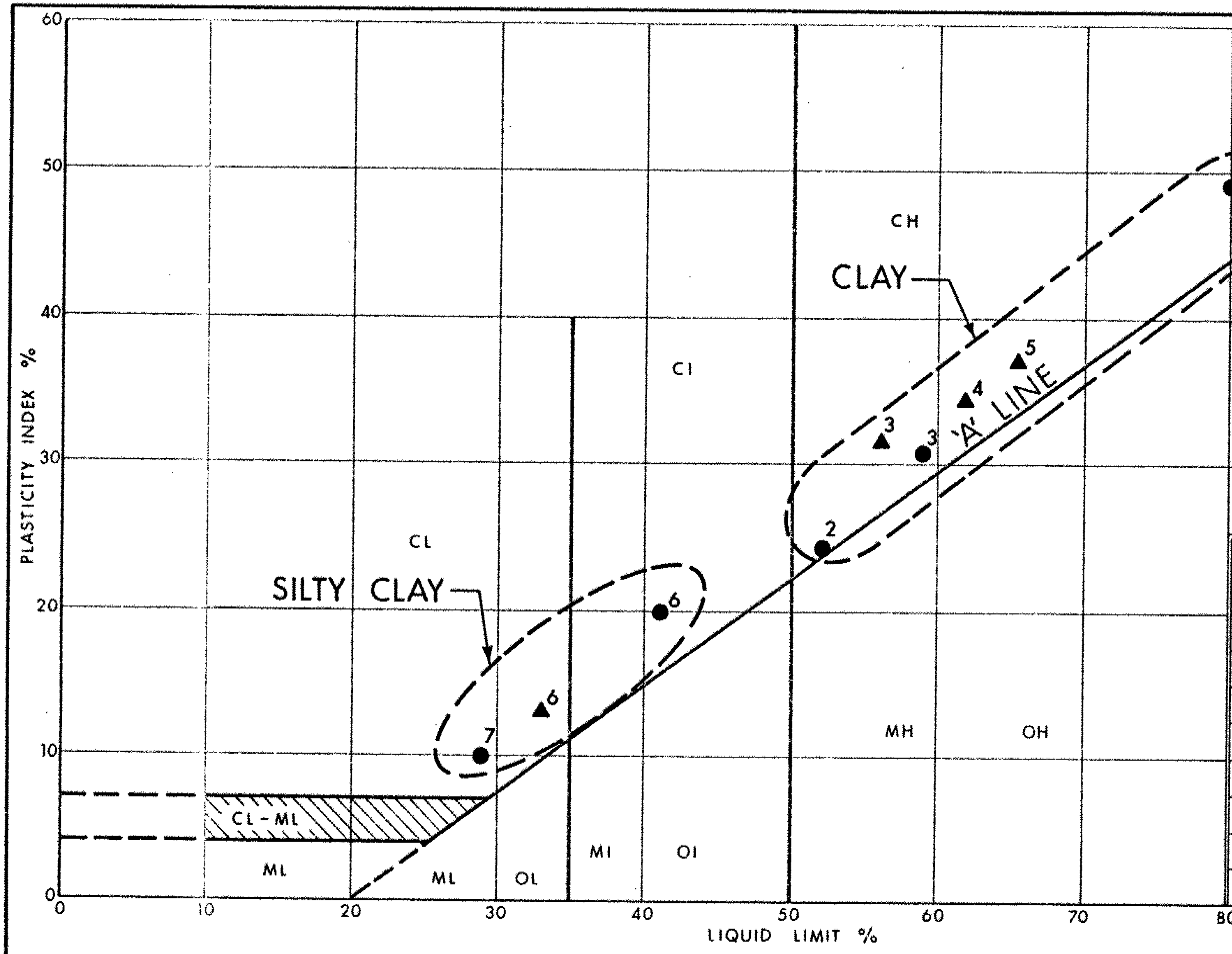
REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+3, x5 : Numbers refer to Sensitivity

15 -◇- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Oct 75, FF-S-21.



LEGEND		
BH	SAMPLE	SYMBOL
1	AS SHOWN	●
4	AS SHOWN	▲

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PLASTICITY CHART

FIG No 2

W.P. 145-74-04

Steven Creek

VOID RATIO–PRESSURE DIAGRAM
CONSOLIDATION TEST

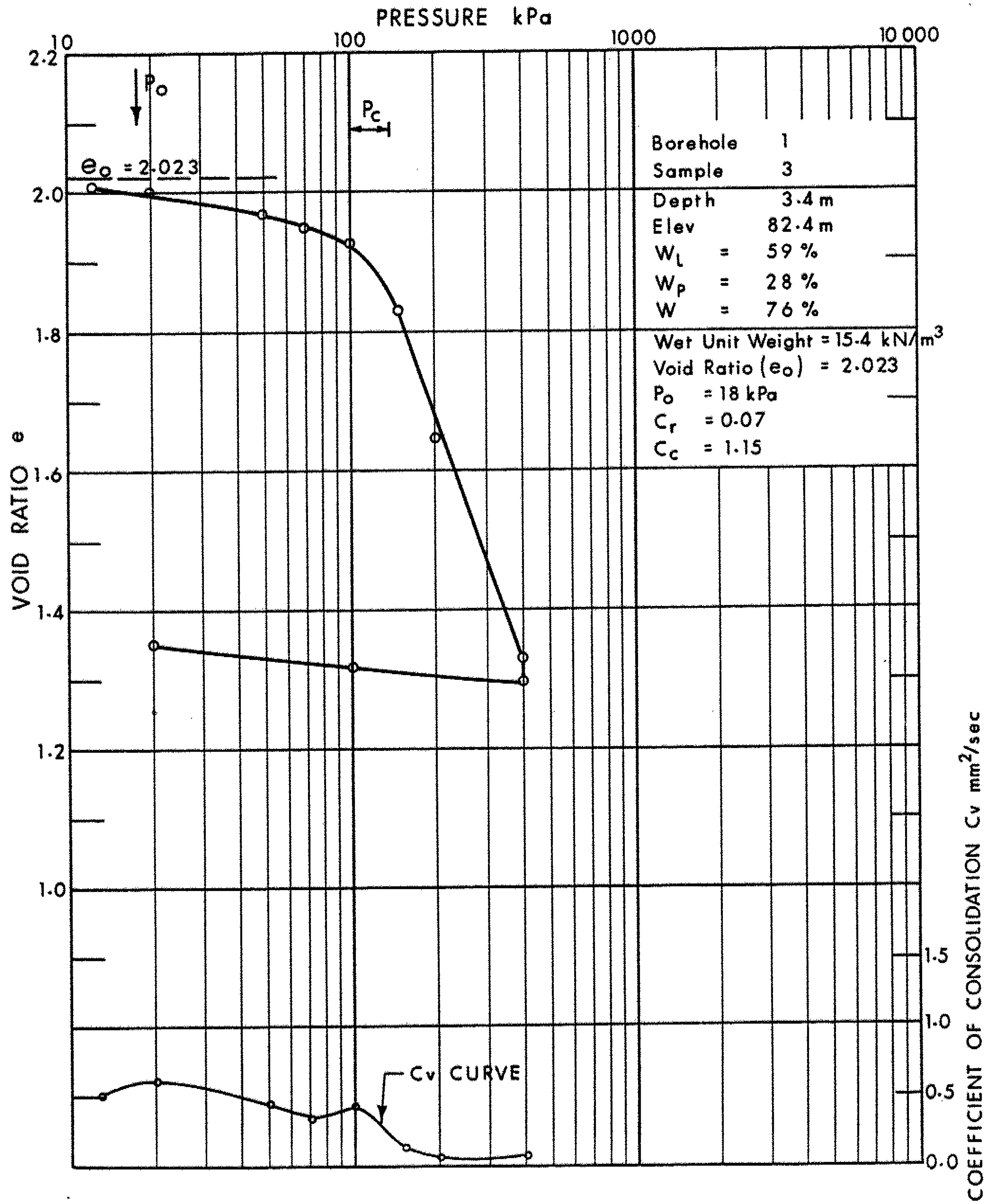
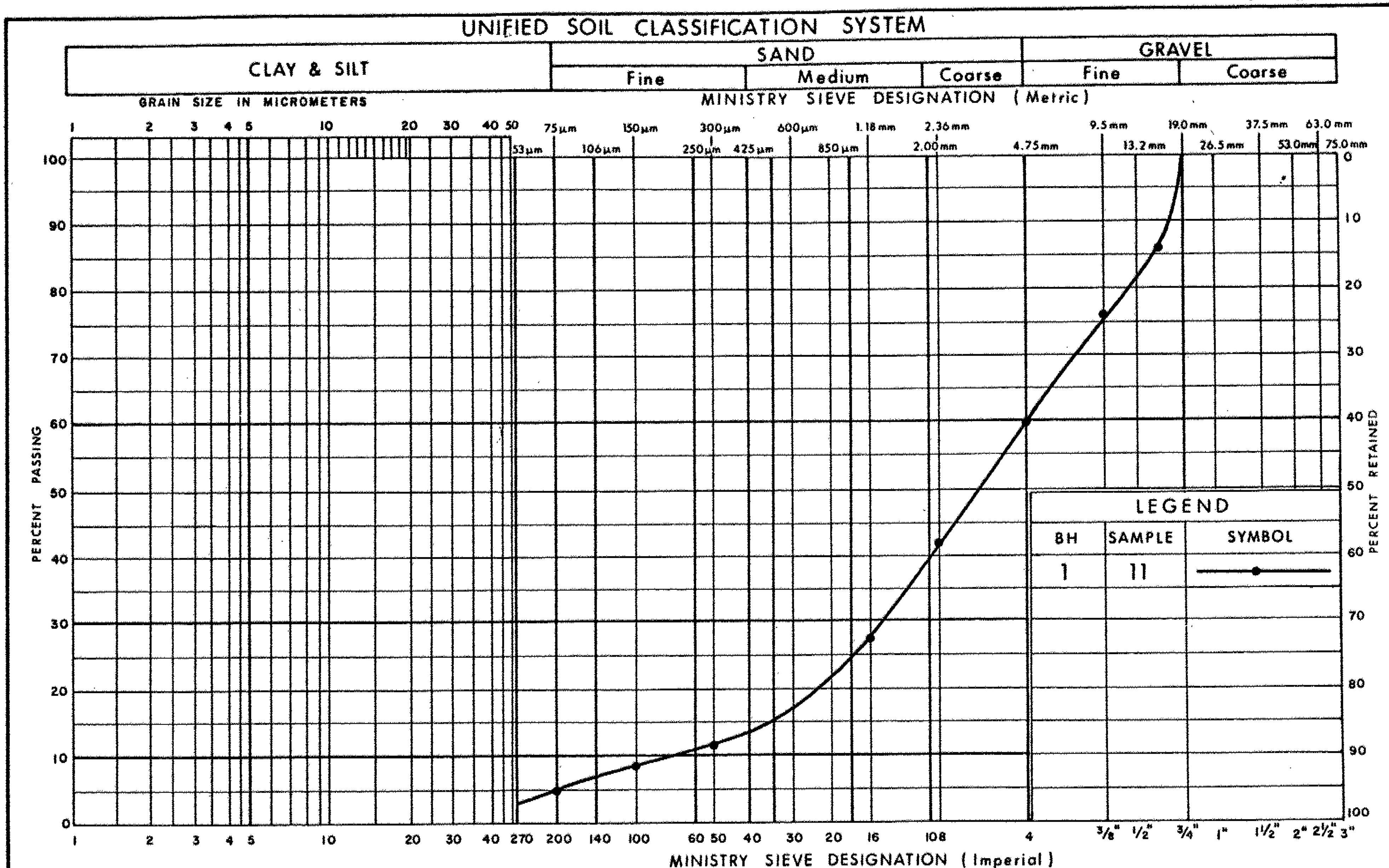


FIG No 3

W P 145–74–04
Steven Creek

78 12 M



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GRAIN SIZE DISTRIBUTION
SAND AND GRAVEL (Glacial Till)

FIG No 4	
W P 145-74-04	
STEVEN CREEK	

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

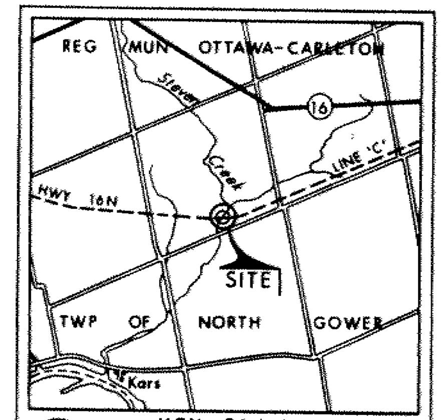
CONT No
WP No 145-74-04

STEVEN CREEK

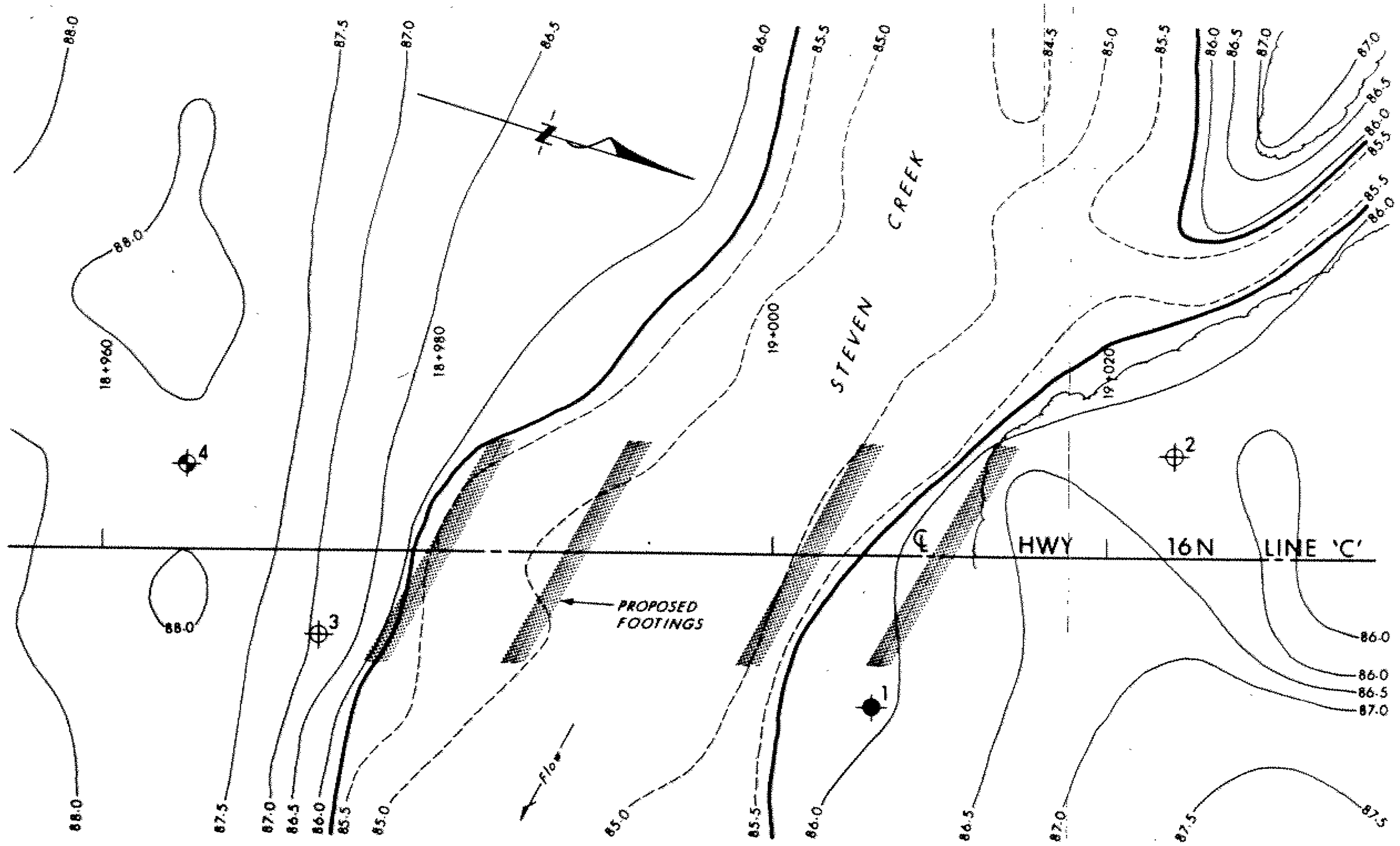
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

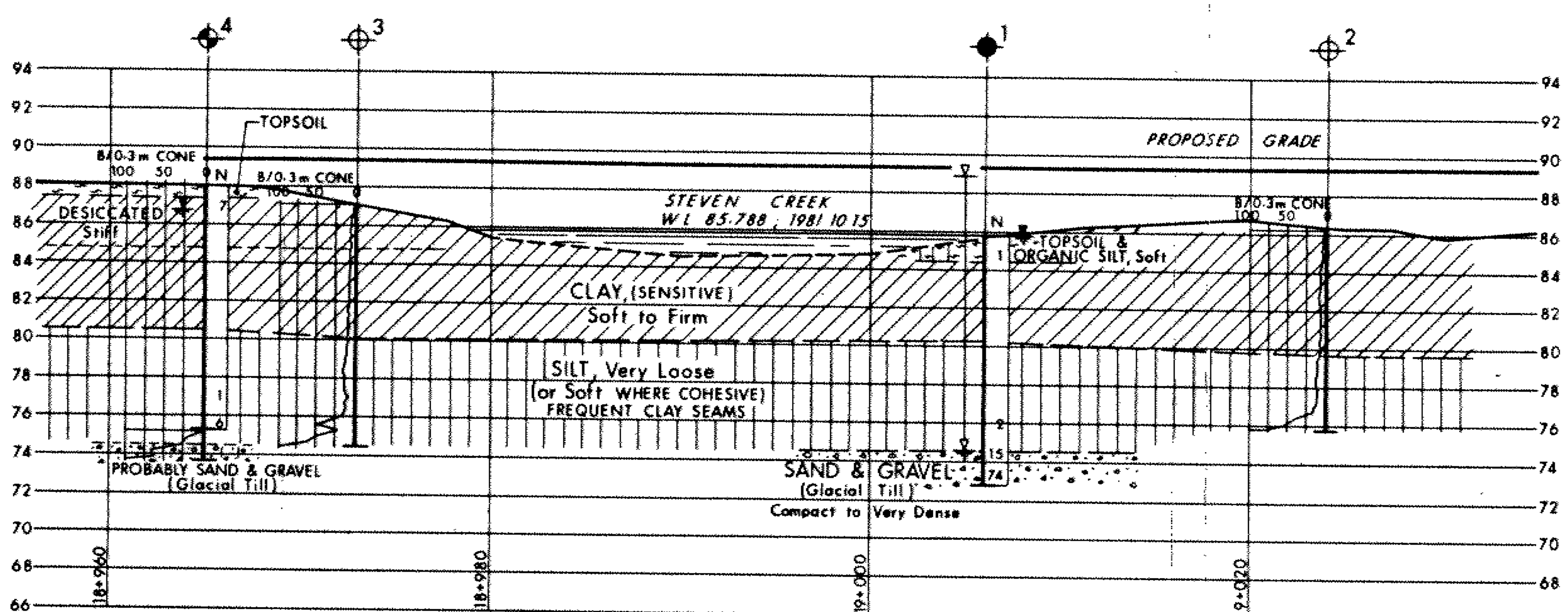
Warnock Hersey Professional Services Ltd.



KEY PLAN
SCALE
1 km 0 1 2 3 km



PLAN
SCALE
4m 2 0 4m



℄ PROFILE HWY 16N LINE 'C'
SCALE
4m 2 0 4m

LEGEND

- ◆ Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation 1981 11
- Artesian Head
- ARTESIAN CONDITIONS
- Artesian Encountered

No	ELEVATION	STATION	OFFSET
1	85.8	19+006.0	9.0 m Rt
2	86.2	19+024.0	6.0 m Lt
3	87.0	18+973.0	5.0 m Rt
4	87.9	18+965.0	5.0 m Lt

NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS | DATE | BY | DESCRIPTION

Geocres No

HWY No 16N			DIST 9
SUBM'D C M	CHECKED	DATE 1981 12 16	SITE 3-356
DRAWN J T	CHECKED	APPROVED	DWG 1457404-A

REF No E-7002-1; 1981 11

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 31G5-190

DIST. 42 REGION

W.P. No. 203-86-02

CONT. No. 99-29

W. O. No.

STR. SITE No. 3-42

HWY. No. 417

LOCATION MAITLAND AVE. UNDERPASS

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

Ministry of Transportation

Ontario

FILE No. ________ **DATE** ________

REMARKS

Ted Downing 1-800-567-6847

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Durisol - Hans Remp

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ADM-G-24 79-06

FOUNDATION INVESTIGATION REPORT

CONTRACT NO. 99-29

Cont. 99-24

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Foundation Investigation Report

for

GWP 203-86-02
Highway 417/Maitland Avenue Bridge Rehabilitation
District 42, Ottawa

INTRODUCTION

This Foundation Investigation Report was prepared as part of the Highway 417/Maitland Avenue Bridge Rehabilitation project (GWP 203-86-02, Agreement No. 9740-7411-4256). The report presents the results of the Foundation Investigation for:

1) Earth retaining structures beneath the Maitland Avenue Bridge; and
2) Foundations for overhead signs.

SITE DESCRIPTION AND GEOLOGY

GWP 203-86-02 is located along a 1.3 km long section of Highway 417 centered around the Maitland Avenue interchange, within the City of Ottawa, Regional Municipality of Ottawa-Carleton (RMOC). The site location is shown on the Key Plan (Drawing No. 11099-1).

This section of Highway 417 is 4-lanes wide in each direction directly beneath the bridge structure with the EBLs and WBLs separated by a concrete barrier.

The existing Maitland Avenue Bridge is a two span continuous steel girder bridge with composite concrete deck. The spans are each approximately 35 m in length. The height of the existing embankments is approximately 4.75 m. The existing bridge abutments are supported on piles, end bearing on bedrock. The front row of piles are battered toward Hwy 417. The foreslopes of both the north and south abutments are at a 2H:1V grade, are protected by stone paving, and extend down to the roadway shoulders.

Drainage along the sections of Hwy 417 immediately east and west of the Maitland Avenue Bridge is provided by highway ditches. Small corrugated steel pipe (CSP) culverts currently link the ditches within the section beneath the bridge structure.

This project lies within the physiographic region known as the Ottawa Valley Clay Plains which is interrupted by ridges of rock and sand.

The native surficial materials in the vicinity of the Maitland Avenue Bridge are Champlain Sea deposits, consisting of silt and clay underlying erosional terraces. Bedrock within this area generally consists of limestone of the Ottawa Formation.

INVESTIGATION PROCEDURE

Field Investigation

The subsurface conditions were investigated through a borehole drilling investigation and laboratory testing.

Prior to drilling the boreholes, their locations were laid out by Jacques Whitford staff and the appropriate utility agencies were contacted to ensure that the site was clear of buried utilities.

Due to the high traffic volumes on Highway 417, the drilling investigation was carried out at night within full lane and ramp closures. The boreholes along the eastbound and westbound lanes were drilled on the nights of August 26th and 27th, respectively. The lane and ramp closures were carried out by Beacon Lite Ltd., in accordance with a traffic control plan submitted to and approved by Mr. John Blaikie of MTO.

Portable light stands were used to provide adequate lighting for the drill crews.

The subsurface conditions along the retaining wall alignment and at the proposed location for the overhead sign foundations were investigated by a total of four (4) boreholes, designated as 98-1 through 98-4. Due to site access restrictions (concrete barriers, ditches and steep slopes), one borehole was put down along both the north and south sides of Hwy 417, between the overhead signs and retaining wall alignments, using portable electrically powered equipment. The remaining two boreholes were put down at the base of the slopes in front of the retaining wall alignments using a truck mounted CME 55 drill rig. The borehole locations are shown on Drawing No. 11099-2, attached.

The boreholes were advanced to refusal on inferred bedrock. Split spoon samples were collected at regular 0.76 m intervals while carrying out Standard Penetration Testing (SPT) (ASTM D1586) and the recovered soil samples were identified in the field by our personnel. The SPT carried out in the boreholes put down with the portable equipment utilized a 20.4 kg hammer with a 0.76 m drop as opposed to a standard 63.5 kg hammer with a 0.76 m drop. The N-values presented on these Borehole Records have been corrected by reducing the field N-value by a factor of 3 to reflect the difference in energy delivered by the hammer during the testing. In-situ shear vane testing was attempted at several locations within the cohesive soil deposits, however, at all locations tested, the undrained shear strength of the soil exceeded the field vane capacity of 150 kPa.

The subsurface conditions are described in detail in the Borehole Records presented in Appendix 1. Geotechnical cross sections are shown on Drawing No 11099-2.

All soil samples recovered during the SPT were stored in moisture proof containers and were returned to our laboratory for detailed classification and testing.

Standpipes were installed within three of the boreholes prior to backfilling. The boreholes were backfilled by replacing (and tamping in layers) the augered material.

The field information is supplemented by soil and bedrock information contained on the Borehole Records from the foundation investigation carried out for the Maitland Avenue Bridge construction (1958). The Borehole Records include stratigraphic descriptions of the overburden soils as well as bedrock.

The borehole locations and elevations were surveyed by McCormick Rankin Corporation's (MRC) survey crew, using a geodetic datum.

Laboratory Testing

All samples returned to the laboratory were subjected to detailed visual classification by a geotechnical engineer. Selected samples were tested for moisture content, grain size distributionand Atterberg Limits. One representative soil sample from each of the retaining wall locations was submitted for pH, sulphate and chloride testing to assess the potential for corrosion of buried steel and the potential for sulphate attack on buried concrete. All soil samples will be stored for a period of one year after issuance of the final report. Unless otherwise directed, the stored samples will be disposed of after this period.

SUBSURFACE CONDITIONS

Subsurface Profile

The subsurface conditions observed in the boreholes are presented in detail on the Borehole Records provided in Appendix 2. The Borehole Records from DWG. No. D 4158-3, produced for the Maitland Avenue Bridge construction (W.P. 930-58) have been reproduced and are also included in Appendix 2. An explanation of the symbols and terms used to describe the Borehole Records is also provided.

Stratigraphic profiles are provided in Drawing No 11099-2. The subsurface conditions observed at the site are generally consistent from borehole to borehole and are described in the following sections.

Embankment Fill

The embankment fill in the vicinity of the proposed retaining walls consists primarily of brown, sand, some silt, trace organics (rootlings) and occasional clayey pockets. Some areas of fill consisting of silty sand and clay, some gravel were also encountered. Typical "N" values obtained during Standard Penetration Testing ranged from 3 to 6, indicating a loose material. The thickness of the embankment fill varies significantly due to the 2H:1V foreslopes. The base of the embankment fill varies from elevation 80.8 m to 82.1 m

Clay

The fill is underlain by a deposit of high plasticity clay. The thickness of the clay deposit varies from in Boreholes 98-1 through 98-4 varied from 1.1 m to 3.0 m. The elevation at the based of the clay varied from 78.5 m to 80.1 m. The clay was brown to greyish brown in colour. The moisture contents of seven samples tested ranged from 29 % to 60 %. Atterberg Limit testing carried out on one representative sample of the clay indicated a Liquid Limit of 60 % and a Plastic Limit of 22 %, indicating an inorganic clay of high plasticity (CH). A hydrometer analysis indicated that 42 % of the clay had a grain size between 5 and 75 microns, indicating a moderate susceptibility to frost heaving (MSFH).

Standard Penetration Test "N" values within the clay varied from 9 to 3, and generally decreased with depth. In-situ shear vane testing was attempted at several locations within the clay deposit, however, at all locations tested, the undrained shear strength of the soil exceeded the field vane capacity of 150 kPa, indicating that the consistency of the clay was very stiff to hard. A thin layer of soft silty clay was identified in one borehole (Borehole No. 5 - located at the eastern centre pier) during the 1958 foundation investigation for the original bridge structure.

Heterogeneous Mixture of Sand, Silt and Gravel (Glacial Till)

A glacial till deposit consisting of a heterogeneous mixture of sand, silt and gravel was encountered beneath the clay deposit in each of the boreholes. The deeper portion of the till deposit in Borehole 98-4 was clayey. At all borehole locations, the glacial till extended to bedrock. Standard Penetration Test "N" values varied from 4 to 23. Refusal to penetration of the split spoon sampler was encountered within the Till deposit in Borehole No. 1. The results of the Standard Penetration Tests indicate that the glacial till is generally in a loose to compact state with some areas of dense till. A grain size distribution analysis carried out on a sample of the till indicated that it contained 25 % gravel, 42 % sand and 33 % silt and clay particles. The moisture contents of five samples tested ranged from 10 % to 29 %

Bedrock

Auger refusal on inferred bedrock was encountered in Boreholes 98-1 through 98-4. Bedrock coring was not carried out during the current investigation. Bedrock coring was carried out for the 1958 geotechnical investigation for the Maitland Avenue Bridge structure. The bedrock elevation at these ten borehole locations varied from 77.1 m to 79.3 m.

The records for Borehole Nos 1 through 6 indicate that bedrock consists of limestone with typical bedding thicknesses ranging from 2 to 3 inches (50 mm to 75 mm). No record of Rock Qaulity Designation (RQD) or unconfined compressive strength was available.

Groundwater

Groundwater levels were measured in the standpipes on September 17,1998, approximately three weeks after the boreholes were drilled. The groundwater levels measured in the standpipes varied from elevation 79.2 m to 80.0 m, (approximately 2 m below the finished grade of Hwy 417). Fluctuations in the groundwater level due to seasonal variations or in response to a particular precipitation event should be anticipated.

CLOSURE

Cont. 99-29

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered at the specific borehole locations and can only be extrapolated to an undefined limited area around these locations. The extent of the limited area depends on the soil and groundwater conditions, as well as the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information.

The drilling equipment used was owned and operated by Marathon Drilling Company Ltd.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

Yours very truly,

JACQUES, WHITFORD LIMITED

Paul Carnaffan, M.Eng., P.Eng.

J.G.A. Raymond Haché, M.Sc., P.Eng.

LICENSED PROFESSIONAL ENGINEER
P. CARNAFFAN
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
J. G. A. R. HACHE
PROVINCE OF ONTARIO

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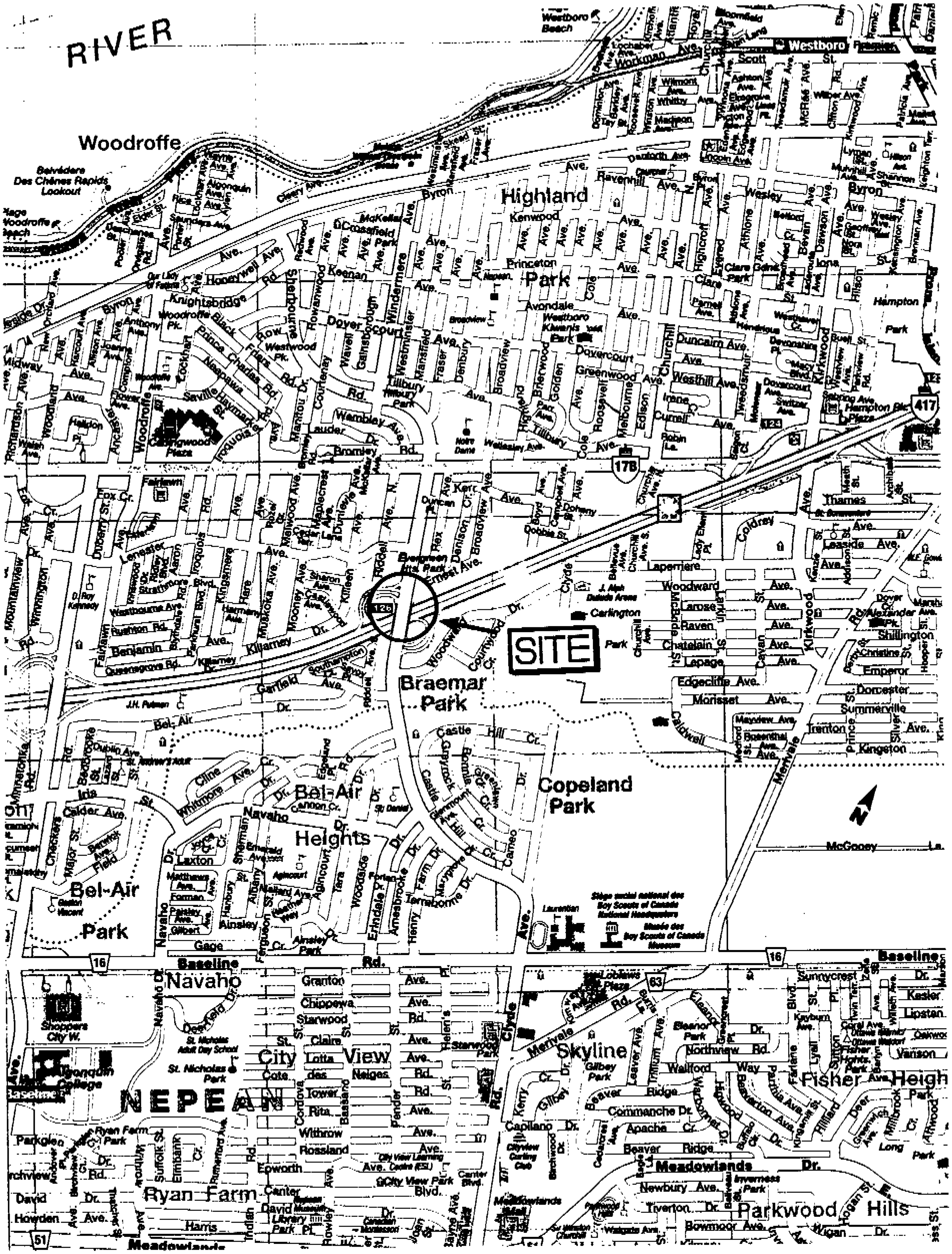
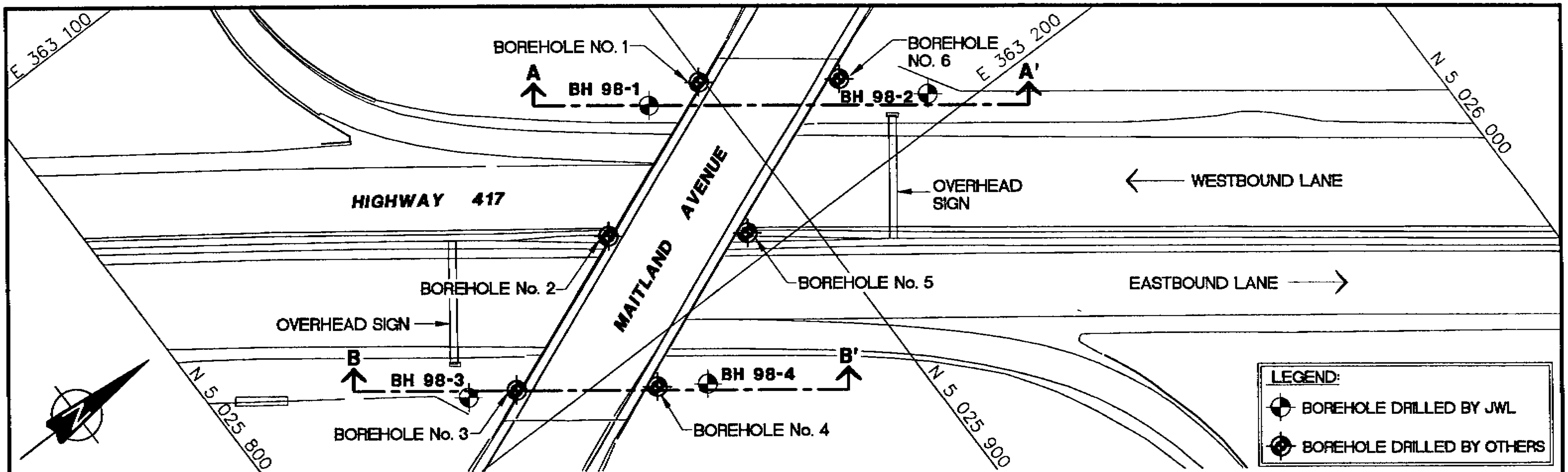


KEY PLAN

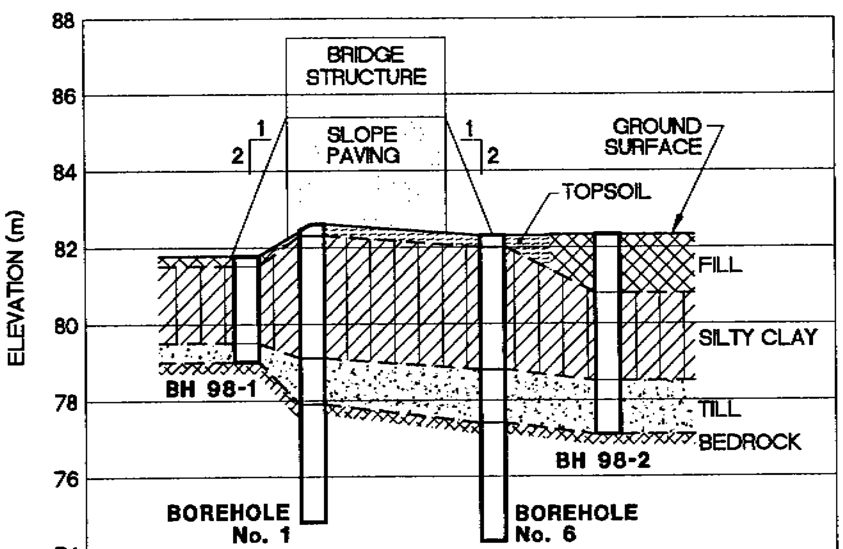
1 : 25 000

9

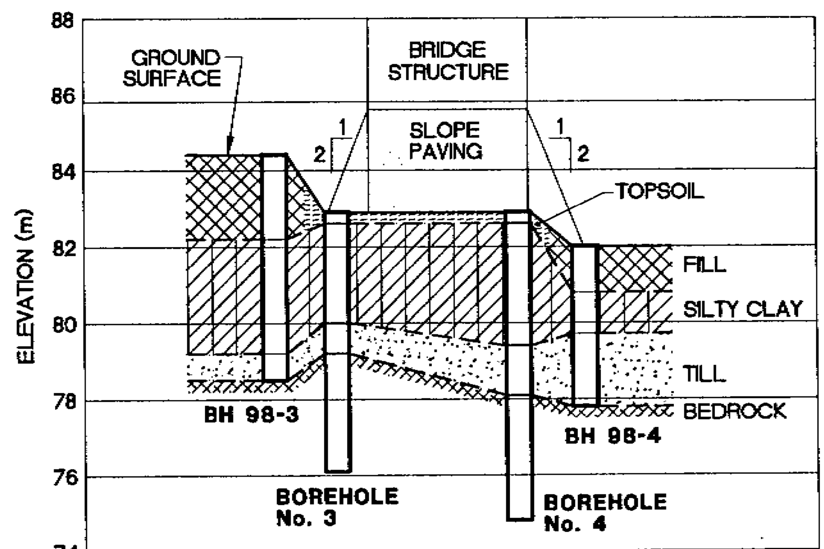
DRAWING No. 11099-1



PLAN VIEW
1 : 1000



CROSS-SECTION A-A'



CROSS-SECTION B-B'

Cont. 99-29

10 McCORMICK RANKIN CORPORATION GWP 203-86-02 HIGHWAY 417/MAITLAND AVENUE BRIDGE REHABILITATION BOREHOLE LOCATION AND CROSS-SECTION PLANS OTTAWA, ONTARIO	Scale: 1 : 1000 HORIZ. 1 : 200 VERT.		Drawing No.: 11099-2	Jacques Whitford
	Date: 98/11/04	Dwn. by: GBB	Appd.:	

Cont. 99-29

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE

e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

RECORD OF BOREHOLE No 98-1

METRIC

W.P. 203-86-02
LOCATION Hwy 417 WBL at Maitland Avenue
ORIGINATED BY LP
DIST 429 HWY 417
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY PC
DATUM Geodetic
DATE 98.08.27 & 98.08.28
CHECKED BY [signature]

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS
82.3 / 0.0	Brown, silty sand, trace organics: FILL	1	SS	5		82	
81.6 / 0.8	Brown, silty clay, some gravel, trace organics: FILL	2	SS	8		81	
80.8 / 1.5	Very stiff, brown, SILTY CLAY, occasional sand seams	3	SS	7		80	
		4	SS	4			51.6
79.3 / 3.0	Stiff	5	SS	3	∇	79	
78.5 / 3.8	Loose to compact, grey, silty sand with gravel, trace clay: TILL	6	SS	7		78	
		7	SS	16			
77.1 / 5.2	End of Borehole						

Auger Refusal on Inferred Bedrock

- standpipe installed

- standard penetration tests carried out using 45 lb hammer and 30 inch freefall

max. field vane capacity was 150 kPa which was exceeded by soil resistance.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 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

$\times^3, \times^3$: Numbers refer to Sensitivity   (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

CONT. 99-29

RECORD OF BOREHOLE No 98-2

METRIC

W.P. 203-86-02 | LOCATION Hwy 417 WBL at Maitland Avenue | ORIGINATED BY LP

DIST 429 HWY 417 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY PC

DATUM Geodetic | DATE 98.08.27 & 98.08.28 | CHECKED BY MH

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
81.8 / 0.0	Dark brown, silty sand: FILL					
81.5 / 0.3	Very stiff, brown, SILTY CLAY	1	SS	3		
						81
		2	SS	9		
						80
		3	SS	7	▽	
79.5 / 2.3	Compact, brown, silty sand, some gravel, trace clay: TILL	4	SS		*REF	
79.0 / 2.8	End of Borehole Auger Refusal on Inferred Bedrock - max. field vane capacity was 150 kPa which was exceeded by soil resistance. *REF = split spoon refusal					79

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 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, ×³: Numbers refer to Sensitivity

20 / 15 ◆ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

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CONT. 99-29

RECORD OF BOREHOLE No 98-3

METRIC

W.P. 203-86-02 LOCATION Hwy 417 EBL at Maitland Avenue ORIGINATED BY LP

DIST 429 HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY PC

DATUM Geodetic DATE 98.08.26 & 98.08.27 CHECKED BY MLP

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 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
84.4 / 0.0	Brown, sand, some silt, trace organics, clayey pockets: FILL		1	SS	6		84				
			2	SS	3		83				
			3	SS	6						
82.1 / 2.3	Very stiff, brown, SILTY CLAY, occasional sand seams		4	SS	8		82				
			5	SS	6		81				
80.6 / 3.8	stiff		6	SS	3		80		44.2		
			7	SS	5				59.9		
79.2 / 5.3	Loose, brown, clayey sand, some silt, trace gravel: TILL		8	SS	7		79				
78.5 / 5.9	End of Borehole Auger Refusal on Inferred Bedrock - max. field vane capacity was 150 kPa which was exceeded by soil resistance.										

×³, ×³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 98-4

METRIC

W.P. 203-86-02 LOCATION Hwy 417 EBL at Maitland Avenue ORIGINATED BY LP

DIST 429 HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY PC

DATUM Geodetic DATE 98.08.26 & 98.08.27 CHECKED BY [signature]

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
82.0 / 0.0	Brown, silty sand: FILL	1	SS	16		82
		2	SS	5		81
80.8 / 1.2	Very stiff to stiff, brown, SILTY CLAY	3	SS	4	∇	80
79.7 / 2.3	Loose, brown-grey, silty sand with gravel, trace clay: TILL	4	SS	8		
79.0 / 3.1	Grey, silty clay,som gravel: TILL	5	SS	4		79
		6	SS		*REF	78
77.8 / 4.2	End of Borehole					

End of Borehole

Auger Refusal on Inferred Bedrock

- standpipe installed

- standard penetartion tests carried out using 45 lb hammer and 30 inch freefall

*REF = split spoon refusal

Plot columns: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, ✕ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 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/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Water content plot annotations: 60.1, 64.6

$\times^3, \times^3$: Numbers refer to Sensitivity; 15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 1

METRIC

W.P. 930-58 LOCATION Bridge No. 3 at Maitland Avenue ORIGINATED BY MTO

DIST 9 HWY 417 BOREHOLE TYPE COMPILED BY MTO

DATUM Geodetic DATE 58.05.09 & 58.05.12 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS
82.6 / 0.0	TOP SOIL				
82.3 / 0.3	HARD, FISSURED, SILTY, BROWNISH GREY CLAY	1	SS	23	
80.8 / 1.9	VERY STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	2	SS	14	
		3	SS	10	
		4	SS	13	
79.1 / 3.5	DENSE SANDY TILL	5	SS	150 / 225 mm	
		6	SS	65 / 75 mm	
77.9 / 4.7	LIMESTONE (drilled)	7	RC		REC = 71%
77.3 / 5.3	LIMESTONE (drilled) bedding thickness 2.5"	8	RC		REC = 89%
76.6 / 6.0	LIMESTONE (drilled) bedding thickness 3"	9	RC		REC = 96%
75.4 / 7.2	LIMESTONE (drilled) bedding thickness 2"	10	RC		REC = 87%
74.8 / 7.8	BOTTOM OF HOLE				

Record of Borehole reproduced from Dwg No. D 4158-3

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (82, 81, 80, 79, 78, 77, 76, 75); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, ✕ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 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/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Groundwater level symbol shown at approx. elevation 82.

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

20 / 15 -⊕- 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

METRIC

W.P. 930-58 LOCATION Bridge No. 3 at Maitland Avenue ORIGINATED BY MTO

DIST 9 HWY 417 BOREHOLE TYPE COMPILED BY MTO

DATUM Geodetic DATE 58.05.15 & 58.05.15 CHECKED BY

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
83.0						
0.0 / 82.7	TOP SOIL					
0.3	HARD, FISSURED, SILTY, BROWNISH GREY CLAY	1	SS		82	
81.6 / 1.4	VERY STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	2	SS		81	
80.8 / 2.2	STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	3	SS			
80.1 / 2.9	LOOSE TILL	4	SS	5	80	
79.3 / 3.7	LIMESTONE (drilled) bedding thickness 2.5"	5	RC		79, 78	REC=93%
77.8 / 5.2	LIMESTONE (drilled) bedding thickness 3"	6	RC		77	REC=100%
76.3 / 6.7	BOTTOM OF HOLE					

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, ✕ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 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/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$\times^3, \times^3$: Numbers refer to Sensitivity   15-20-5-10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

METRIC

W.P. 930-58 LOCATION Bridge No. 3 at Maitland Avenue ORIGINATED BY MTO

DIST 9 HWY 417 BOREHOLE TYPE COMPILED BY MTO

DATUM Geodetic DATE 58.05.13 & 58.05.13 CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%)
82.9					
82.6 / 0.0	TOP SOIL				
81.6 / 0.3	HARD, FISSURED, SILTY, BROWNISH GREY CLAY	1	SS		
80.8 / 1.4	VERY STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	2	SS		
80.0 / 2.2	STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	3	SS		
79.3 / 2.9	LOOSE TILL	4	SS		
78.1 / 3.7	LIMESTONE (drilled) bedding thickness 2"	5	RC		REC = 83%
76.9 / 4.8	LIMESTONE (drilled) bedding thickness 2" to 3"	6	RC		REC = 100%
76.1 / 6.0	LIMESTONE (drilled) bedding thickness 3"	7	RC		REC = 86%
6.8	BOTTOM OF HOLE				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (82, 81, 80, 79, 78, 77); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 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/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$\times^3$, $\times^3$: Numbers refer to Sensitivity; 20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

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CONT. 99-29

RECORD OF BOREHOLE No 4

METRIC

W.P. 930-58 LOCATION Bridge No. 3 at Maitland Avenue ORIGINATED BY MTO

DIST 9 HWY 417 BOREHOLE TYPE COMPILED BY MTO

DATUM Geodetic DATE 58.05.12 & 58.05.12 CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
82.9							
0.0	TOP SOIL						
82.6 0.3	HARD, FISSURED, SILTY, BROWNISH GREY CLAY	1	SS		▽	82	
81.5 1.4	VERY STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	2	SS			81	
80.7 2.2	STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	3	SS			80	
		4	SS				
79.4 3.5	LOOSE TILL	5	SS	7		79	
78.3 4.6	DENSE TILL						
78.1 4.8	LIMESTONE (drilled) bedding thickness 2" some vertical seams	6	RC			78, 77	REC = 81%
76.6 6.3	LIMESTONE (drilled) bedding thickness 2"	7	RC				REC = 60%
76.0 6.9	LIMESTONE (drilled) bedding thickness 2"	8	RC			76, 75	REC = 80%
74.8 8.1	BOTTOM OF HOLE						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 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 — GR SA SI CL

$\times^3, \times^3$: Numbers refer to Sensitivity   15-20-5-10 (%) STRAIN AT FAILURE

CONT. 99-29

RECORD OF BOREHOLE No 5

METRIC

W.P. 930-58 LOCATION Bridge No. 3 at Maitland Avenue ORIGINATED BY MTO

DIST 9 HWY 417 BOREHOLE TYPE COMPILED BY MTO

DATUM Geodetic DATE 58.05.14 & 58.05.14 CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)
82.7						
0.0	TOP SOIL					
82.4 / 0.3	HARD, FISSURED, SILTY, BROWNISH GREY CLAY	1	SS		▽	
81.3 / 1.4	VERY STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	2	SS			
		3	SS			
79.8 / 2.9	STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	4	SS			
79.0 / 3.7	SOFT, VERY SILTY, GREY CLAY	5	SS	1 /450mm		
78.4 / 4.3	DENSE TILL	6	SS	23		
		7	SS	63 /75mm		
78.0 / 4.7	LIMESTONE (drilled)	8	RC			REC = 60%
77.5 / 5.2	LIMESTONE (drilled) bedding thickness 3"	9	RC			REC = 33%
76.6 / 6.1	LIMESTONE (drilled) bedding thickness 3" one 80 degree joint in core break	10	RC			REC = 87%
75.0 / 7.7	LIMESTONE (drilled) bedding thickness 3"	11	RC			REC = 100%
74.5 / 8.2	BOTTOM OF HOLE					

Column headings also printed: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (82, 81, 80, 79, 78, 77, 76, 75); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 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/m^3; GR SA SI CL

$\times^3, \times^3$: Numbers refer to Sensitivity   20 15 5 10 (%) STRAIN AT FAILURE

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CONT. 99-29

RECORD OF BOREHOLE No 6

METRIC

W.P. 930-58 | LOCATION Bridge No. 3 at Maitland Avenue | ORIGINATED BY MTO

DIST 9 HWY 417 | BOREHOLE TYPE | COMPILED BY MTO

DATUM Geodetic | DATE 58.05.13 & 58.05.14 | CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
82.3 / 0.0	TOP SOIL						
82.0 / 0.3	HARD, FISSURED, SILTY, BROWNISH GREY CLAY	1	SS		∇	82	
81.0 / 1.4	VERY STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	2	SS			81	
80.2 / 2.2	STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	3	SS			80	
		4	SS			79	
78.8 / 3.5	LOOSE TILL	5	SS	7			
78.2 / 4.1	DENSE TILL	6	SS	33 /150mm		78	
		7	SS	44 /75mm			
77.4 / 5.0	LIMESTONE (drilled) bedding thickness 2"	8	RC			77	REC=93%
76.3 / 6.0	LIMESTONE (drilled) bedding thickness 3"	9	RC			76	REC=94%
74.9 / 7.4	LIMESTONE (drilled) bedding thickness 3"	10	RC			75	REC=95%
74.3 / 8.0	BOTTOM OF HOLE						

Column headers also include: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 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/m^3; GR SA SI CL.

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

GEOCRES No
3165-190

FOUNDATION INVESTIGATION REPORT

GWP 203-86-02
HIGHWAY 417/MAITLAND AVENUE BRIDGE REHABILITATION

DISTRICT 42, OTTAWA

MINISTRY OF TRANSPORTATION ONTARIO

SUBMITTED TO

McCORMICK RANKIN CORPORATION

BY

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PROJECT NO. 11099

FOUNDATION INVESTIGATION REPORT

TO

McCORMICK RANKIN CORPORATION

ON

GWP 203-86-02
HIGHWAY 417/MAITLAND AVENUE BRIDGE REHABILITATION

DISTRICT 42, OTTAWA
MINISTRY OF TRANSPORTATION ONTARIO

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November 6, 1998

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FOUNDATION INVESTIGATION REPORT

for

GWP 203-86-02
Highway 417/Maitland Avenue Bridge Rehabilitation
District 42, Ottawa

1.0 INTRODUCTION

This report presents the results of a geotechnical foundation investigation carried out as part of the Highway 417/Maitland Avenue Bridge Rehabilitation project (GWP 203-86-02, Agreement No. 9740-7411-4256). This bridge rehabilitation project includes the bridge rehabilitation work as well as pavement widenings of the Highway 417 lanes, construction of earth retaining structures beneath the bridge and relocation of two overhead sign foundations, as well as intersection improvements at the Ramp E-N/S and Maitland Avenue intersection.

This Foundation Investigation Report presents geotechnical recommendations for the design and construction of:

1) Earth retaining structures beneath the Maitland Avenue Bridge; and
2) Foundations for overhead signs.

A separate Pavement Design Report (PDR) has been prepared for the pavement widenings and the Ramp E-N/S intersection works.

The work was carried out in general accordance with Schedule 6 of the TPM proposal submission.

This report has been prepared specifically and solely for the project described herein. It contains factual information obtained from this investigation pertaining to the subsurface conditions.

2.0 SITE DESCRIPTION AND GEOLOGY

GWP 203-86-02 is located along a 1.3 km long section of Highway 417 centered around the Maitland Avenue interchange. The site location is shown on the Key Plan provided in Appendix 1 (Drawing No. 11099-1).

This section of Highway 417 is 4-lanes wide in each direction directly beneath the bridge structure with the EBLs and WBLs separated by a concrete barrier.

The existing bridge abutments are supported on piles, end bearing on bedrock. The front row of piles are battered toward Hwy 417. The foreslopes of both the north and south abutments are at a 2H:1V grade, are protected by stone paving, and extend down to the roadway shoulders.

Drainage along the sections of Hwy 417 immediately east and west of the Maitland Avenue Bridge is provided by highway ditches. Small corrugated steel pipe (CSP) culverts currently link the ditches within the section beneath the bridge structure.

This project lies within the physiographic region known as the Ottawa Valley Clay Plains which is interrupted by ridges of rock and sand.

The native surficial materials in the vicinity of the Maitland Avenue Bridge are Champlain Sea deposits, consisting of silt and clay underlying erosional terraces. Bedrock within this area generally consists of limestone of the Ottawa Formation.

3.0 PROCEDURE

3.1 Field Investigation

The site soil conditions were investigated through a borehole drilling investigation and laboratory testing.

Prior to drilling the boreholes, their locations were laid out by Jacques Whitford staff and the appropriate utility agencies were contacted to ensure that the site was clear of buried utilities.

Due to the high traffic volumes on Highway 417, the drilling investigation was carried out at night within full lane and ramp closures. The boreholes along the eastbound and westbound lanes were drilled on the nights of August 26th and 27th, respectively. The lane and ramp closures were carried out by Beacon Lite Ltd., in accordance with a traffic control plan submitted to and approved by Mr. John Blaikie of MTO.

Portable light stands were used to provide adequate lighting for the drill crews.

The soil conditions along the retaining wall alignment and at the proposed location for the overhead sign foundations were investigated by a total of four (4) boreholes, designated as 98-1 through 98-4. Due to site access restrictions (concrete barriers, ditches and steep slopes), one borehole was put down along both the north and south sides of Hwy 417, between the overhead signs and retaining wall alignments, using portable electrically powered equipment. The remaining two boreholes were put down at the base of the slopes in front of the retaining wall alignments using a truck mounted CME 55 drill rig. The borehole locations are shown on Drawing No. 11099-2, attached.

The boreholes were advanced to refusal on inferred bedrock. Split spoon samples were collected at regular 2½ foot intervals while carrying out Standard Penetration Testing (SPT) (ASTM D1586) and the recovered soil samples were identified in the field by our personnel. The SPT carried out in the boreholes put down with the portable equipment utilized a 45 pound hammer with a 30 inch drop as opposed to a standard 140 lb hammer with a 30 inch drop. The N-values presented on these Borehole Records have been corrected by reducing the field N-value by a factor of 3 to reflect the difference in energy delivered by the hammer during the testing. In-situ shear vane testing was attempted at several locations within the cohesive soil deposits, however, at all locations tested, the undrained shear strength of the soil exceeded the field vane capacity of 150 kPa.

The subsurface conditions are described in detail in the Borehole Records presented in Appendix 1. Geotechnical cross sections are shown on Drawing No 11099-2.

All soil samples recovered during the SPT were stored in moisture proof containers and were returned to our laboratory for detailed classification and testing.

Standpipes were installed within three of the boreholes prior to backfilling. The boreholes were backfilled by replacing (and tamping in layers) the augered material.

The field information is supplemented by soil and bedrock information contained on the Borehole Records from the foundation investigation carried out for the Maitland Avenue Bridge construction (1958). These Borehole Records include stratigraphic descriptions of the overburden soils as well as bedrock.

3.2 Survey

The borehole locations and elevations were surveyed by McCormick Rankin Corporation's (MRC) survey crew, using a geodetic datum.

3.3 Laboratory Testing

All samples returned to the laboratory were subjected to detailed visual classification by a geotechnical engineer. Selected samples were tested for moisture content, grain size distribution and Atterberg Limits. One representative soil sample from each of the retaining wall locations was submitted for pH, sulphate and chloride testing to assess the potential for corrosion of buried steel and the potential for sulphate attack on buried concrete. All soil samples will be stored for a period of one year after issuance of the final report. Unless otherwise directed, the stored samples will be disposed of after this period.

4.0 RESULTS OF THE INVESTIGATION

4.1 Subsurface Profile

The subsurface conditions observed in the boreholes are presented in detail on the Borehole Records provided in Appendix 2. The Borehole Records from DWG. No. D 4158-3, produced for the Maitland Avenue Bridge construction (W.P. 930-58) have been reproduced and are also included in Appendix 2. An explanation of the symbols and terms used to describe the Borehole Records is also provided. Stratigraphic profiles are provided in Drawing No 11099-2. The subsurface conditions observed at the site are generally consistent from borehole to borehole and are described in the following sections.

4.1.1 Embankment Fill

The embankment fill within the slopes beside the slope paving consists of silty sand with varying amounts of gravel, occasional rootlings and clayey pockets.

4.1.2 Silty Clay

The fill is underlain by a silty clay deposit which extends to elevation 78.5 m to 80.1 m. The silty clay generally has a hard to stiff consistency and is brown to greyish-brown in colour. A thin layer of soft silty clay was identified in Borehole No. 5, located at the eastern centre pier. The moisture contents of seven samples tested ranged from 29 % to 60 %. Atterberg Limit testing carried out on one representative sample of the silty clay indicated a Liquid Limit of 60 % and a Plastic Limit of 22 %. A hydrometer analysis indicated that 42 % of the silty clay had a grain size between 5 and 75 microns, indicating a moderate susceptibility to frost heaving (MSFH).

4.1.3 Glacial Till

A glacial till deposit was encountered in each of the boreholes below the silty clay. The glacial till extended to bedrock and was in a loose to dense state. A grain size distribution analysis carried out on a sample of the till indicated that it contained 25 % gravel, 42 % sand and 33 % silt and clay particles. The moisture contents of five samples tested ranged from 10 % to 29 %

4.2 Groundwater

Groundwater levels were measured in the standpipes on September 17,1998. The measured groundwater levels varied from elevation 79.2 m to 80.0 m, (approximately 2 m below the finished grade of Hwy 417). Fluctuations in the groundwater level due to seasonal variations or in response to a particular precipitation event should be anticipated.

4.3 Bedrock

Refusal on inferred bedrock was encountered in Boreholes 98-1 through 98-4. Bedrock coring was carried out for the 1958 geotechnical investigation for the Maitland Avenue Bridge structure. The bedrock elevation at these ten borehole locations varied from 77.1 m to 79.3 m. The records for Borehole Nos 1 through 6 indicate that bedrock consists of limestone with typical bedding thicknesses ranging from 2 to 3 inches (50 mm to 75 mm).

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 Proposed Development

Pavement widenings will be carried out beneath the Maitland Avenue Bridge to accommodate temporary construction detours as well as future widening of the Highway. It is understood that Highway 417 near Maitland Avenue will be widened by one lane in each direction, likely within the next 5 to 7 years.

To create space for the proposed pavement widenings, the existing slope paving and approach fill in front of both the north and south bridge abutments will be cut back and retained. The exact limits of the proposed retaining structures were not known at the time of the investigation. In addition, the existing overhead signs located just before the Maitland Avenue bridge in both the EBL and WBL will be widened. This will require moving the outside leg of each sign to a new foundation located approximately 4 m further out.

It is understood that a tieback retaining wall system, incorporating H-piles and rock anchors, is the preferred design alternative to hold back the approach fill in front of the bridge abutments. In addition, new foundations are to be constructed for support of the outside legs of the overhead signs near the Maitland Avenue Bridge.

Based on information provided by others, the existing bridge abutments are supported on end bearing piles, and the proposed work discussed herein is not to impact on these foundations.

5.2 Geotechnical Assessment

A tie-back retaining wall system is compatible with the geotechnical conditions at the site. The final design of the soldier pile and rock anchor locations will need to consider the location of the piles supporting the existing abutments.

The overhead signs may be founded on spread footings founded within the native silty clay or on bored piles.

The frost penetration depth to be used in all design work at this site is 1.8 m. The proposed retaining wall location will result in a reduction in frost protection to the underside of the existing abutments - as little as 0.85 m cover will be provided to the south abutment. Therefore, frost protection of the existing abutments will be required.

5.3 Geotechnical Recommendations for Earth Retaining Structures

5.3.1 Lateral Earth Pressures

Computation of earth pressures should be in accordance with Section 6-7 of the OHBDC 3rd Edition. For rigidly tied structures, where the wall is allowed to rotate less than 0.1 % of the wall height, the at-rest earth pressure should be used for design. Typically, for a tie-back retaining wall system, minimum excavation and backfilling is carried out behind the retaining wall, therefore lateral earth pressures acting on the walls should be based on soil parameters for the existing embankment fill. The following unfactored soil parameters may be used for design.

Soil Parameter	Existing Fill	
	1.5H : 1V Backslope	**2H : 1V Backslope**
Bulk Unit Weight, γ (kN/m^3)	20.5	20.5
Effective Friction Angle, ϕ	30°	30°
Rankine Coefficient of Active Earth Pressure (K_a)	0.59	0.54
Coefficient of Earth Pressure at Rest (K_o)	0.90	0.85

Slope paving should be placed to protect the steep (1.5H:1V) backslope. Backslopes with a vegetated cover should be made no steeper than 2:1. In addition, due to the steep backslope beneath the bridge structure, it is recommended that the upper 600 mm (minimum) of the backslope material consist of OPSS Granular A. This detail is shown in Drawing No. 11099-5, in Appendix 3.

5.3.2 Lateral Resistance of Vertical Piles

The lateral resistance of vertical piles should be determined in accordance with Section 6-9.8 of the Ontario Highway Bridge Design Code (OHBDC) 3rd Edition. The resistance provided by the soil may be determined in accordance with the methods described by B.B. Broms in the following papers, or by other appropriate methods:

Broms, B.B., 1964. "Lateral Resistance of Piles in Cohesive Soils". J. of Soil Mech. and Found .Div., ACSE, vol. 90, SM2: 27-63

Broms, B.B., 1964. "Design of Laterally Loaded Piles in Cohesionless Soils". J. of Soil Mech. and Found .Div., ACSE, vol. 90, SM3: 123-156

Broms, B.B., 1965. "Design of Laterally Loaded Piles". J. of Soil Mech. and Found .Div., ACSE, vol. 91, SM3: 79-99

The following unfactored design parameters may be used for the design of laterally loaded piles:

Material	Unit Weight (kN/m^3)	Effective Friction Angle, ϕ	Rankine Passive Earth Pressure, K_p	Undrained Shear Strength, C_u (kPa)
Very Stiff Silty Clay	17.0	n/a	n/a	125
Stiff Silty Clay	17.0	n/a	n/a	75
Till	20.5	30°	3.0	n/a
Pavement Structure	22.0	35°	3.7	n/a

Where the upper material along the passive face consists of a drained pavement structure abutting the tie-back retaining wall, at least 800 mm thick, the passive pressure within the full frost penetration depth may be used, but should be reduced by a factor of 0.5, as depicted in Drawing No. 11099-6 in Appendix 3. Where the surficial soils along the passive face of the piles consist of native material or random fill, the passive pressure within the frost penetration depth of 1.8 m should be neglected when determining the lateral resistance.

5.3.3 Vertical Resistance of Piles

Tip elevations for end-bearing steel soldier piles are expected to range from 77 to 79 m.

No downdrag forces are expected at the proposed retaining wall locations since no additional fill placement is expected.

Due to limited clear height beneath the existing bridge deck, it is anticipated that pile driving will not be possible. Rather, it is anticipated that the piles will be installed using a vibratory hammer. Prior to installation of piles, it will be necessary to locate existing batter piles. It is anticipated that the clearance between existing piles and the new piles will be in the order of 375 mm. It is therefore recommended that the exposed portions of the existing piles be at least 600 mm in length so that the plumb direction may be determined using an electronic level such as a "Smart Level" or "Smart Tool" in order to project an approximate tip location prior to installing the new piles.

The following design parameters are recommended for steel piles installed using a vibratory hammer at this site:

Pile Type	Factored Resistance at ULS (kN)	Resistance at SLS (kN)
W250x49	380	270
W310x60	450	320
W360x64	490	350

Pile installation should be monitored by a qualified and experienced inspector. Attempts should be made in all cases to install the piles to a dense stratum.

5.3.4 Tieback (rock) Anchors

Grouted rock anchors should be used to tie back the wall. The following recommendations are provided for the design of grouted rock anchors:

- A rock to grout bond stress of 500 kPa (ULS) may be used for holes grouted with non-shrink grout having a minimum compressive strength of 30 Mpa.
- The minimum fixed anchor length (i.e. the length over which the rock to grout bond stress is developed) should be no less than 3 m.
- The minimum anchor spacing should be 900 mm centre to centre.
- To ensure against the possibility of a rock mass failure, the following design parameters may be used:
  - submerged unit weight = 15.2 kN/m^3
  - a 60° (appex angle) failure cone with the appex located at the midpoint of the bonded length
- The interaction between cones must be included in the overall stability analysis

All rock anchors should be proof loaded to 150 % of the design load. The minimum free anchor length normally required for stressing of the anchor is 1 to 2 m. This length may vary depending on the type of anchor and stressing equipment. The anchor designer/installer should be consulted to verify this requirement.

The free anchor length may be grouted, after the proof load test to provide corrosion protection to the anchor tendon.

5.3.5 Frost Protection

Proposed retaining structures will need to be protected against frost action. For the proposed concrete wall facing, this can be achieved by one of the following three methods:

1. Extending the base of the concrete facing to a depth of 1.8 m below grade.

2. Constructing a drained granular pad beneath the concrete panels. The granular pad should consist of free draining material which extends to 1.8 m below ground surface. If this option is selected, the details will be provided.

3. The use of extruded polystyrene insulation to provide the equivalent of 1.8 m of soil cover as protection. For design purposes at this site, 25 mm of insulation will provide frost protection equivalent to 500 mm of soil cover. Design details will need to be reviewed by the geotechnical consultant.

Existing foundations which will lose their soil protection will also need to be reviewed by the geotechnical consultant once the insulation details are developed.

5.4 Foundation Recommendations - Overhead Signs

The following design parameters may be used for footings as wide as 3 m:

Factored Bearing Resistance at ULS	300 kPa
Bearing Resistance at SLS	200 kPa

The bearing resistance at SLS is based on a maximum allowable settlement of 25 mm.

The effect of inclined loads on the bearing resistance should be accounted for as per Section 6-8.4.2 of the OHBDC, 3rd Edition.

All spread footings should be protected from frost action by a minimum soil cover of 1.8 m or equivalent insulation.

The foundations for the overhead signs may also be founded on bored piles. Laterally loaded bored piles may be designed using the methods and design parameters provided in Section 5.3.2 of this report.

5.5 General Construction Recommendations

Site Grading and Preparation

All organic soils, and other deleterious materials must be removed from beneath spread footings. Where deleterious materials are encountered, the material should be excavated, wasted and replaced with earth fill. The lateral extent of such excavation should include all deleterious material within an imaginary line drawn at an angle of 1 horizontal to 1 vertical, downward and away from the vertical edges of the culvert, to the competent native soil.

Stripping of deleterious materials should be inspected by geotechnical personnel to ensure that all unsuitable materials are removed prior to placement of concrete or Select Subgrade Material (SSM).

If required for grading purposes, earth fill should consist of Select Subgrade Material (SSM), placed in lifts no greater than 300 mm and compacted to at least 95 % Standard Proctor Maximum Dry Density (SPMDD).

Excavation and Backfill

Side slopes for open cut excavations should conform to Occupational Health and Safety Act (OHSA) regulations. Excavation side slopes should be inspected regularly for signs of instability and flattened as required. Alternatively excavation side slopes may be supported.

The existing granular fill is considered a Type 3 soil, in accordance with the OHSA, and excavations should therefore be carried out using side slopes no steeper than 1H:1V from the base of the excavation.

The very stiff silty clay would be considered a Type 2 soil, in accordance with the OHSA, and excavations should therefore be carried out using side slopes no steeper than 1H:1V from 1.2 m above the base of the excavation.

Where backfill material is required behind the retaining wall lagging, it is recommended that a clean material such as silica sand or pea gravel be used. This material should be tamped in place.

Dewatering

Excavations carried out above the water table are expected to receive minor groundwater inflow due to surface run-off and precipitation. Dewatering may be carried out using conventional sump and pump methods. Auger holes for bored piles will likely extend beneath the water table. Significant groundwater inflow may be expected if the auger hole penetrates the silty clay into the underlying glacial till or if sand seams within the silty clay are penetrated. Tremie techniques may be required for placement of concrete under these conditions.

Cement Type and Corrosion Protection

Two representative soil sample were submitted to Accutest Laboratories in Nepean, Ontario, for analysis of pH and water soluble sulphate and chloride, in order to determine cement type and reinforcing steel protection requirements.

The water soluble sulphate results were both 0.03 %. Results below 0.10 % are considered to represent a low degree of exposure to sulphate attack and therefore a normal Type 10 Portland cement should be suitable for use in concrete mixtures for this site.

The pH test results were 7.5 and 7.6. Test results between 5.5 and 9.0 are not considered to represent an environment overly conducive to corrosion of steel in contact with the site soils or groundwater. Water soluble chloride levels greater than 0.25 % are also an indication of an environment conducive to corrosion. The test results were between 0.149 % and 0.216 %, however, it should be noted that deicing carried out along this section of Highway 417 consists of a 100 % salt spread and therefore elevated chloride levels should be expected immediately adjacent to the future edge of roadway, possibly creating an environment favourable for corrosion of steel members such as piles and reinforcing steel.

6.0 CLOSURE

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review our recommendations when the drawings and specifications are complete.

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered at the specific borehole locations and can only be extrapolated to an undefined limited area around these locations. The extent of the limited area depends on the soil and groundwater conditions, as well as the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information and its effects on the above conclusions.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

Yours very truly,

JACQUES, WHITFORD LIMITED

Paul Carnaffan

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PROVINCE OF ONTARIO

Paul Carnaffan, M.Eng., P.Eng.

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J. G. A. R. HACHE
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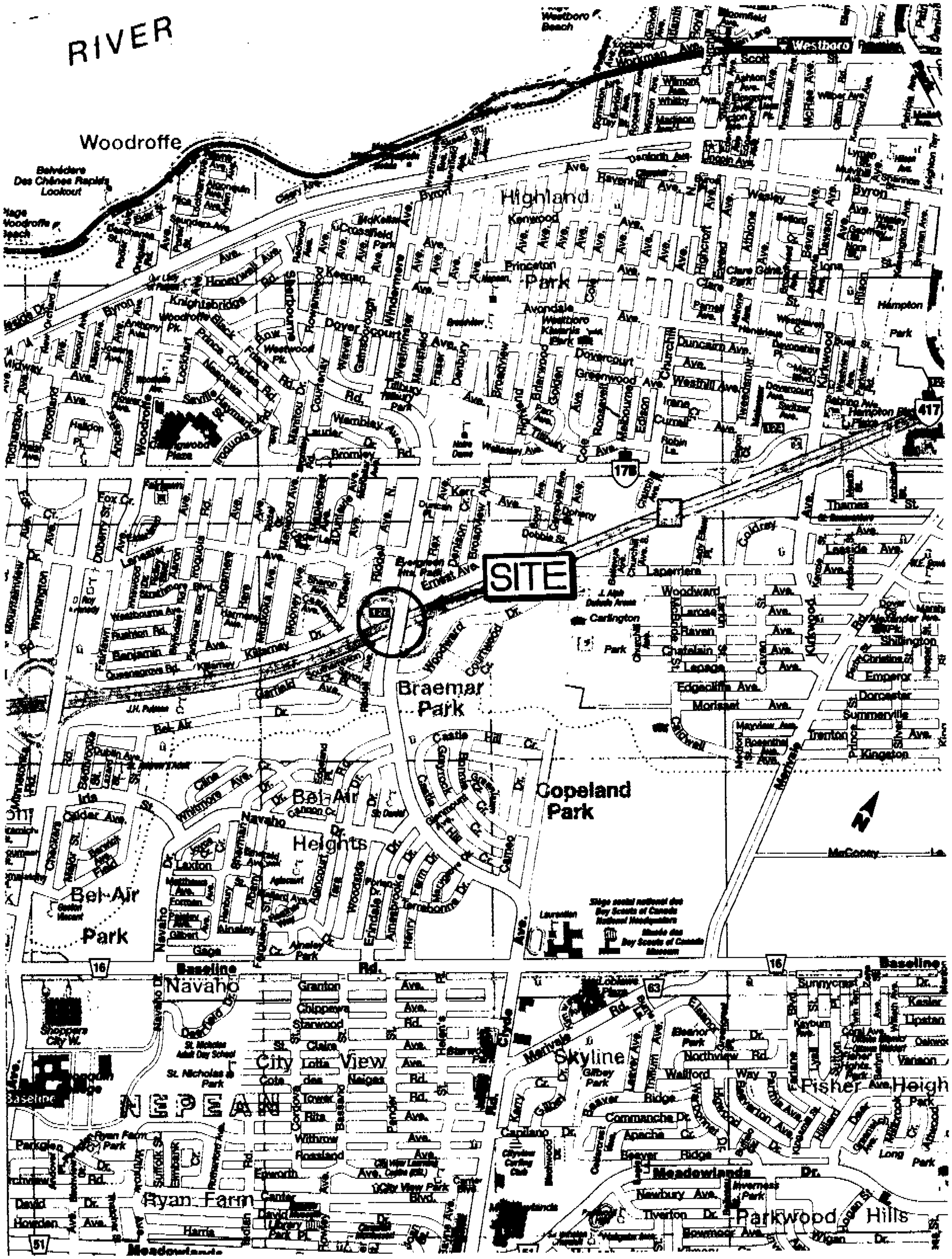
J.G.A. Raymond Haché, M.Sc., P.Eng.

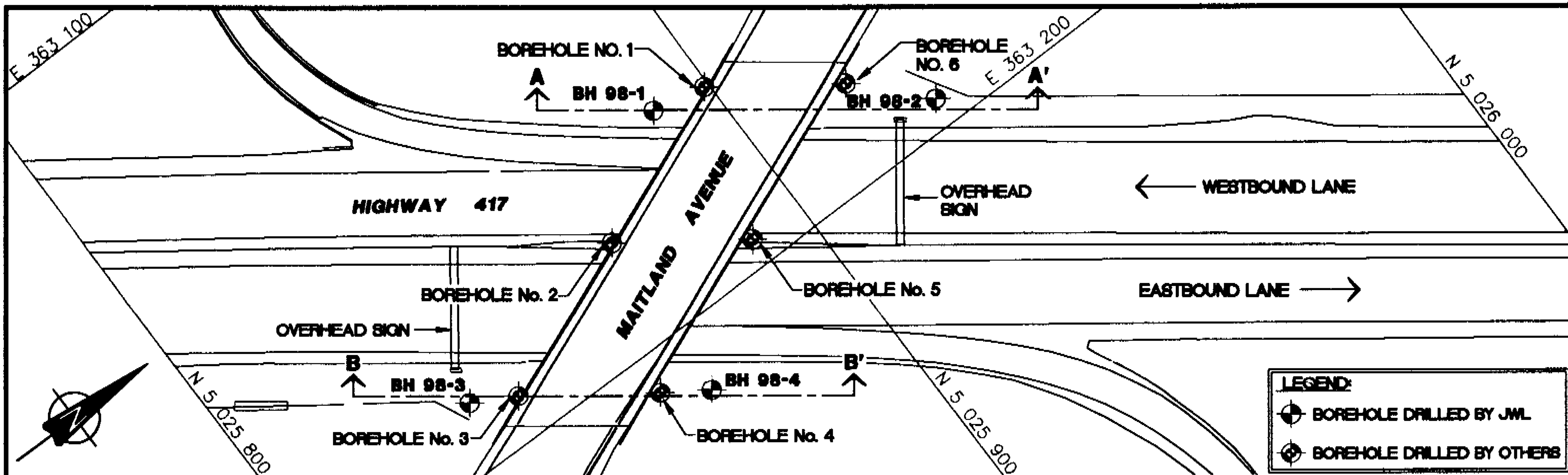
APPENDIX 1



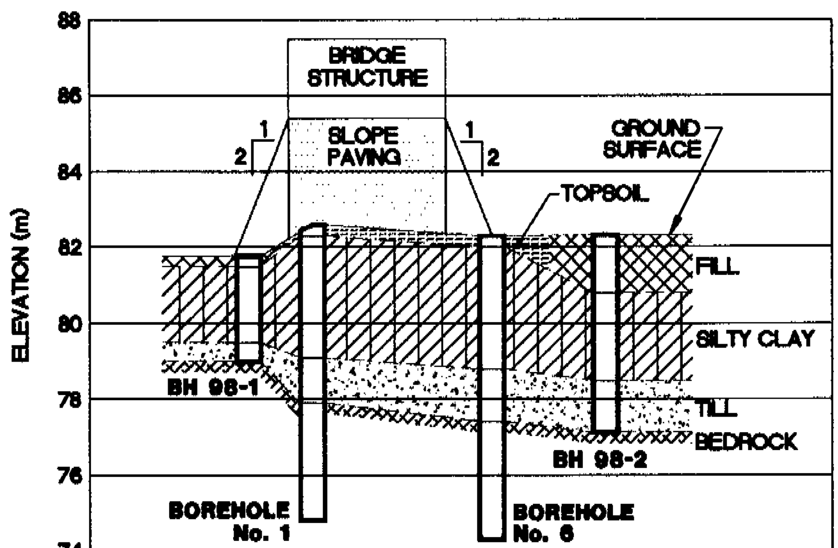
KEY PLAN

1 : 25 000

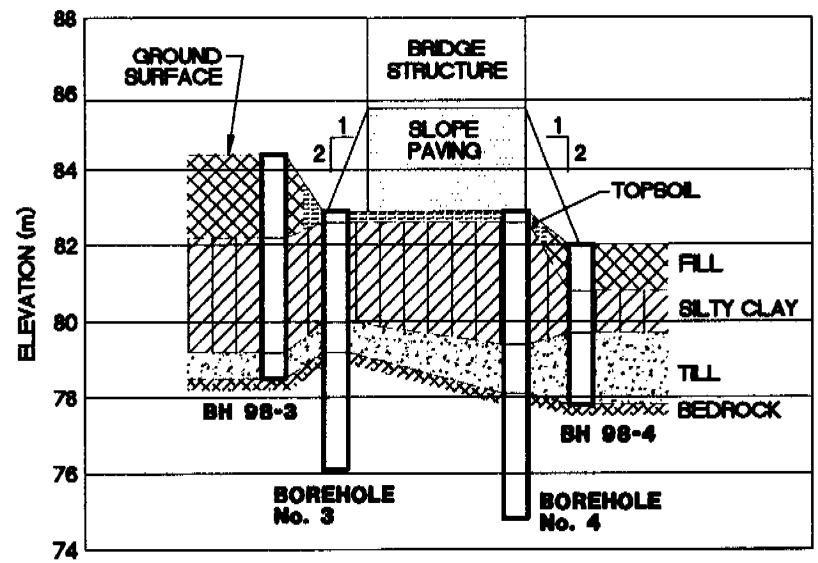
DRAWING No. 11099-1



PLAN VIEW
1:1000



CROSS-SECTION A-A'



CROSS-SECTION B-B'

McCORMICK RANKIN CORPORATION
GWP 203-86-02
HIGHWAY 417/MAITLAND AVENUE BRIDGE REHABILITATION
BOREHOLE LOCATION AND CROSS-SECTION PLANS
OTTAWA, ONTARIO

Scale: 1:1000 HORIZ. 1:200 VERT.	Drawing No.: 11099-2
Date: 98/11/04	Dwn. by: GBB
Appd.: PC	

Jacques Whitford

APPENDIX 2

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Topsoil	-	mixture of soil and humus capable of supporting good vegetative growth
Peat	-	fibrous aggregate of visible and invisible fragments of decayed organic matter
Till	-	unstratified glacial deposit which may range from clay to boulders
Fill	-	any materials below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	-	having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure
Varved	-	composed of regular alternating layers of silt and clay
Stratified	-	composed of alternating successions of different soil types, e.g. silt and sand
Layer	-	>75 mm
Seam	-	2 mm to 75 mm
Parting	-	< 2 mm
Well Graded	-	having wide range in grain sizes and substantial amounts of all intermediate particle sizes
Uniformly Graded	-	predominantly of one grain size

Terminology describing soils on the basis of grain size and plasticity is based on the Unified Soil Classification System (USCS) (ASTM D-2488). The classification excludes particles larger than 76 mm (3 inches). This system provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%

The standard terminology to describe cohesionless soils includes the compactness (formerly "relative density"), as determined by laboratory test or by the Standard Penetration Test 'N' - value.

Relative Density	'N' Value	Compactness %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by insitu vane tests, penetrometer tests, unconfined compression tests, or occasionally by standard penetration tests.

Consistency	Undrained Shear Strength		'N' Value
	kips/sq.ft.	kPa	
Very Soft	<0.25	<12.5	<2
Soft	0.25-0.5	12.5-25	2-4
Firm	0.5-1.0	25-50	4-8
Stiff	1.0-2.0	50-100	8-15
Very Stiff	2.0-4.0	100-200	15-30
Hard	>4.0	>200	>30

ROCK DESCRIPTION

Rock Quality Designation (RQD)

The classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. RQD was originally intended to be done on NW core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from *in situ* fractures.

RQD	ROCK QUALITY
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

Terminology describing rock mass:

Spacing (mm)	Bedding, Laminations, Bands	Discontinuities
2000-6000	*Very Thick*	*Very Wide*
600-2000	*Thick*	*Wide*
200-600	*Medium*	*Moderate*
60-200	*Thin*	*Close*
20-60	*Very Thin*	*Very Close*
<20	*Laminated*	*Extremely Close*
<6	*Thinly Laminated*	

Strength Classification	Uniaxial Compressive Strength (MPa)
Very Low	1-25
Low	25-50
Medium	50-100
High	100-200
Very High	>200

Terminology describing weathering:

Slight - Weathering limited to the surface of major discontinuities. Typically iron stained.

Moderate - Weathering extends throughout rock mass. Rock is not friable.
High - Weathering extends throughout rock mass. Rock is friable.

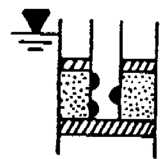
STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:

Boulders Cobbles Gravel | Sand | Silt | Clay | Organics | Asphalt | Concrete | Fill | Igneous Bedrock | Meta-morphic Bedrock | Sedi-mentary Bedrock

WATER LEVEL MEASUREMENT

Borehole or Standpipe



Piezometer

SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
PS	Piston sample
BS	Bulk sample
WS	Wash sample

HQ, NQ, BQ, etc. Rock core samples obtained with the use of standard size diamond drilling bits.

N - VALUE

Numbers in this column are the results of the Standard Penetration Test: the number of blows of a 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (305 mm) into the soil. For split spoon samples where insufficient penetration was achieved and 'N' values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75).

OTHER TESTS

S	Sieve analysis
G_s	Specific gravity of soil particles
k	Permeability (cm/sec)
	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer
H	Hydrometer analysis
γ	Unit weight
C	Consolidation
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct shear
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$; the index corrected to a reference diameter of 50 mm)

RECORD OF BOREHOLE No 98-1

METRIC

W.P. 203-86-02 LOCATION Hwy 417 WBL at Maitland Avenue ORIGINATED BY LP

DIST 429 HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY PC

DATUM Geodetic DATE 98.08.27 & 98.08.28 CHECKED BY PC

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)
82.3 / 0.0	Brown, silty sand, trace organics: FILL	1	SS	5		82	
81.6 / 0.8	Brown, silty clay, some gravel, trace organics: FILL	2	SS	8		81	
80.8 / 1.5	Very stiff, brown, SILTY CLAY, occasional sand seams	3	SS	7		80	
		4	SS	4			61.6
79.3 / 3.0	Stiff	5	SS	3	▽	79	
78.5 / 3.8	Loose to compact, grey, silty sand with gravel, trace clay: TILL	6	SS	7		78	
		7	SS	16			
77.1 / 5.2	End of Borehole						

Auger Refusal on Inferred Bedrock

- standpipe installed

- standard penetration tests carried out using 45 lb hammer and 30 inch freefall

max. field vane capacity was 150 kPa which was exceeded by soil resistance.

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 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

$\times^3, \times^3$: Numbers refer to Sensitivity; 20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 98-2

METRIC

W.P. 203-86-02 LOCATION Hwy 417 WBL at Maitland Avenue ORIGINATED BY LP

DIST 429 HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY PC

DATUM Geodetic DATE 98.08.27 & 98.08.28 CHECKED BY PC

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (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
81.8											
0.0	Dark brown, silty sand: FILL										
81.5 / 0.3	Very stiff, brown, SILTY CLAY		1	SS	3						
							81				
			2	SS	9						
							80				
			3	SS	7	∇					
79.5 / 2.3	Compact, brown, silty sand, some gravel, trace clay: TILL										
79.0			4	SS		*REF	79				
2.8	End of Borehole										

Auger Refusal on Inferred Bedrock

- max. field vane capacity was 150 kPa which was exceeded by soil resistance.

*REF=split spoon refusal

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 98-3

METRIC

W.P. 203-86-02	LOCATION Hwy 417 EBL at Maitland Avenue		ORIGINATED BY LP
DIST 429	HWY 417	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY PC
DATUM Geodetic	DATE 98.08.26 & 98.08.27		CHECKED BY pc

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%) / REMARKS
84.4 / 0.0	Brown, sand, some silt, trace organics, clayey pockets: FILL	1	SS	6	
		2	SS	3	
		3	SS	6	
82.1 / 2.3	Very stiff, brown, SILTY CLAY, occasional sand seams	4	SS	8	
		5	SS	6	
80.6 / 3.8	stiff	6	SS	3	44.1
		7	SS	5	59.9
79.2 / 5.3	Loose, brown, clayey sand, some silt, trace gravel: TILL	8	SS	7	
78.5 / 5.9	End of Borehole Auger Refusal on Inferred Bedrock - max. field vane capacity was 150 kPa which was exceeded by soil resistance.				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (84, 83, 82, 81, 80, 79); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 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

$\times^3, \times^3$: Numbers refer to Sensitivity   20 / 15 – 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 98-4

METRIC

W.P. 203-86-02 | LOCATION Hwy 417 EBL at Maitland Avenue | ORIGINATED BY LP

DIST 429 HWY 417 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY PC

DATUM Geodetic | DATE 98.08.26 & 98.08.27 | CHECKED BY PC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)
82.0 / 0.0	Brown, silty sand: FILL	1	SS	16		82	
		2	SS	5		81	
80.8 / 1.2	Very stiff to stiff, brown, SILTY CLAY	3	SS	4	▽	80	60.1
79.7 / 2.3	Loose, brown-grey, silty sand with gravel, trace clay: TILL	4	SS	8			
79.0 / 3.1	Grey, silty clay,som gravel: TILL	5	SS	4		79	
		6	SS		*REF	78	
77.8 / 4.2	End of Borehole						

Auger Refusal on Inferred Bedrock

- standpipe installed

- standard penetartion tests carried out using 45 lb hammer and 30 inch freefall

*REF = split spoon refusal

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (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

$\times^3, \times^3$: Numbers refer to Sensitivity — (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No 1

METRIC

W.P. 930-58 LOCATION Bridge No. 3 at Maitland Avenue ORIGINATED BY MTO

DIST 9 HWY 417 BOREHOLE TYPE COMPILED BY MTO

DATUM Geodetic DATE 58.05.09 & 58.05.12 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)
82.6 / 0.0	TOP SOIL					
82.3 / 0.3	HARD, FISSURED, SILTY, BROWNISH GREY CLAY	1	SS	23	∇ (approx. el. 81.9)	
80.8 / 1.9	VERY STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	2	SS	14		
		3	SS	10		
		4	SS	13		
79.1 / 3.5	DENSE SANDY TILL	5	SS	150 / 225 mm		
		6	SS	65 / 75 mm		
77.9 / 4.7	LIMESTONE (drilled)	7	RC			REC = 71%
77.3 / 5.3	LIMESTONE (drilled) bedding thickness 2.5"	8	RC			REC = 89%
76.6 / 6.0	LIMESTONE (drilled) bedding thickness 3"	9	RC			REC = 96%
75.4 / 7.2	LIMESTONE (drilled) bedding thickness 2"	10	RC			REC = 87%
74.8 / 7.8	BOTTOM OF HOLE					

Record of Borehole reproduced from Dwg No. D 4158-3

Chart columns: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (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; GR SA SI CL. ELEVATION SCALE: 82, 81, 80, 79, 78, 77, 76, 75.

$✖^3, \times^3$ Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

METRIC

W.P. 930-58 LOCATION Bridge No. 3 at Maitland Avenue ORIGINATED BY MTO

DIST 9 HWY 417 BOREHOLE TYPE COMPILED BY MTO

DATUM Geodetic DATE 58.05.15 & 58.05.15 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
83.0 / 0.0	TOP SOIL						
82.7 / 0.3	HARD, FISSURED, SILTY, BROWNISH GREY CLAY	1	SS			82	
81.6 / 1.4	VERY STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	2	SS			81	
80.8 / 2.2	STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	3	SS			80	
80.1 / 2.9	LOOSE TILL	4	SS	5			
79.3 / 3.7	LIMESTONE (drilled) bedding thickness 2.5"	5	RC			79, 78	REC = 93%
77.8 / 5.2	LIMESTONE (drilled) bedding thickness 3"	6	RC			77	REC = 100%
76.3 / 6.7	BOTTOM OF HOLE						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, ● QUICK TRIAXIAL, ✖ FIELD VANE, × LAB VANE; 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

GR SA SI CL

×³, ×³: Numbers refer to Sensitivity

15-20-5-10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

METRIC

W.P. 930-58 LOCATION Bridge No. 3 at Maitland Avenue ORIGINATED BY MTO

DIST 9 HWY 417 BOREHOLE TYPE COMPILED BY MTO

DATUM Geodetic DATE 58.05.13 & 58.05.13 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
82.9							
0.0	TOP SOIL						
82.6 / 0.3	HARD, FISSURED, SILTY, BROWNISH GREY CLAY	1	SS			82	
81.6 / 1.4	VERY STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	2	SS			81	
80.8 / 2.2	STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	3	SS				
80.0 / 2.9	LOOSE TILL	4	SS			80	
79.3 / 3.7	LIMESTONE (drilled) bedding thickness 2"	5	RC			79	REC=83%
78.1 / 4.8	LIMESTONE (drilled) bedding thickness 2" to 3"	6	RC			78	REC=100%
76.9 / 6.0	LIMESTONE (drilled) bedding thickness 3"	7	RC			77	REC=86%
76.1 / 6.8	BOTTOM OF HOLE						

Column headings also printed: SOIL PROFILE; SAMPLES; STRAT PLOT; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 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/m^3; GR SA SI CL.

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

(%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 4

METRIC

W.P. 930-58 LOCATION Bridge No. 3 at Maitland Avenue ORIGINATED BY MTO

DIST 9 HWY 417 BOREHOLE TYPE COMPILED BY MTO

DATUM Geodetic DATE 58.05.12 & 58.05.12 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
82.9							
0.0 / 82.6	TOP SOIL						
0.3	HARD, FISSURED, SILTY, BROWNISH GREY CLAY	1	SS		▽	82	
81.5 / 1.4	VERY STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	2	SS			81	
80.7 / 2.2	STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	3	SS			80	
		4	SS				
79.4 / 3.5	LOOSE TILL	5	SS	7		79	
78.3 / 4.6	DENSE TILL						
78.1 / 4.8	LIMESTONE (drilled) bedding thickness 2" some vertical seams	6	RC			78, 77	REC = 81%
76.6 / 6.3	LIMESTONE (drilled) bedding thickness 2"	7	RC				REC = 60%
76.0 / 6.9	LIMESTONE (drilled) bedding thickness 2"	8	RC			76, 75	REC = 80%
74.8 / 8.1	BOTTOM OF HOLE						

Other column headings: STRAT PLOT; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 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; GR SA SI CL

$\times^3$, $\times^3$: Numbers refer to Sensitivity   20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 5

METRIC

W.P. 930-58 LOCATION Bridge No. 3 at Maitland Avenue ORIGINATED BY MTO

DIST 9 HWY 417 BOREHOLE TYPE COMPILED BY MTO

DATUM Geodetic DATE 58.05.14 & 58.05.14 CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
82.7							
0.0	TOP SOIL						
82.4 0.3	HARD, FISSURED, SILTY, BROWNISH GREY CLAY	1	SS		▽	82	
81.3 1.4	VERY STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	2	SS			81	
		3	SS			80	
79.8 2.9	STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	4	SS				
79.0 3.7	SOFT, VERY SILTY, GREY CLAY	5	SS	1	450mm	79	
78.4 4.3	DENSE TILL	6	SS	23			
78.0		7	SS	63	75mm	78	
4.7	LIMESTONE (drilled)	8	RC				REC=60%
77.5 5.2	LIMESTONE (drilled) bedding thickness 3"	9	RC			77	REC=33%
76.6 6.1	LIMESTONE (drilled) bedding thickness 3" one 80 degree joint in core break	10	RC			76	REC=87%
75.0 7.7	LIMESTONE (drilled) bedding thickness 3"	11	RC			75	REC=100%
74.5 8.2	BOTTOM OF HOLE						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 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/m^3

x^3, $\times^3$ Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 6

METRIC

W.P. 930-58 LOCATION Bridge No. 3 at Maitland Avenue ORIGINATED BY MTO

DIST 9 HWY 417 BOREHOLE TYPE COMPILED BY MTO

DATUM Geodetic DATE 58.05.13 & 58.05.14 CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)
82.3 / 0.0	TOP SOIL					
82.0 / 0.3	HARD, FISSURED, SILTY, BROWNISH GREY CLAY	1	SS		▽	
81.0 / 1.4	VERY STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	2	SS			
80.2 / 2.2	STIFF, FISSURED, SILTY, BROWNISH GREY CLAY	3	SS			
		4	SS			
78.8 / 3.5	LOOSE TILL	5	SS	7		
78.2 / 4.1	DENSE TILL	6	SS	33	/150mm	
		7	SS	44	/75mm	
77.4 / 5.0	LIMESTONE (drilled) bedding thickness 2"	8	RC			REC = 93%
76.3 / 6.0	LIMESTONE (drilled) bedding thickness 3"	9	RC			REC = 94%
74.9 / 7.4	LIMESTONE (drilled) bedding thickness 3"	10	RC			REC = 95%
74.3 / 8.0	BOTTOM OF HOLE					

Elevation scale: 82, 81, 80, 79, 78, 77, 76, 75

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30

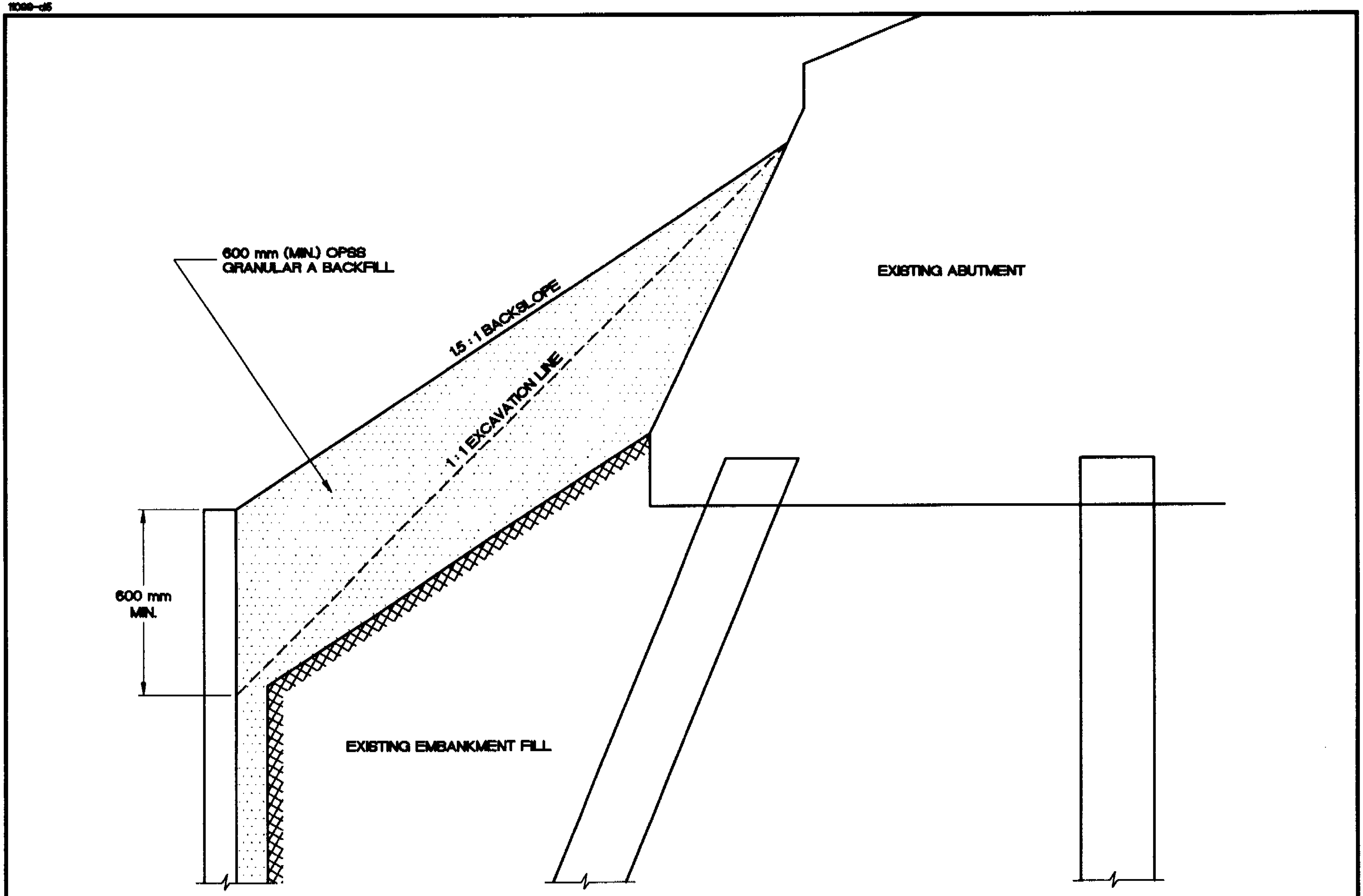
UNIT WEIGHT γ kN/m³

GR SA SI CL

$\times^3, \times^3$: Numbers refer to Sensitivity 15 20 5 10 (%) STRAIN AT FAILURE

APPENDIX 3

11099-d5

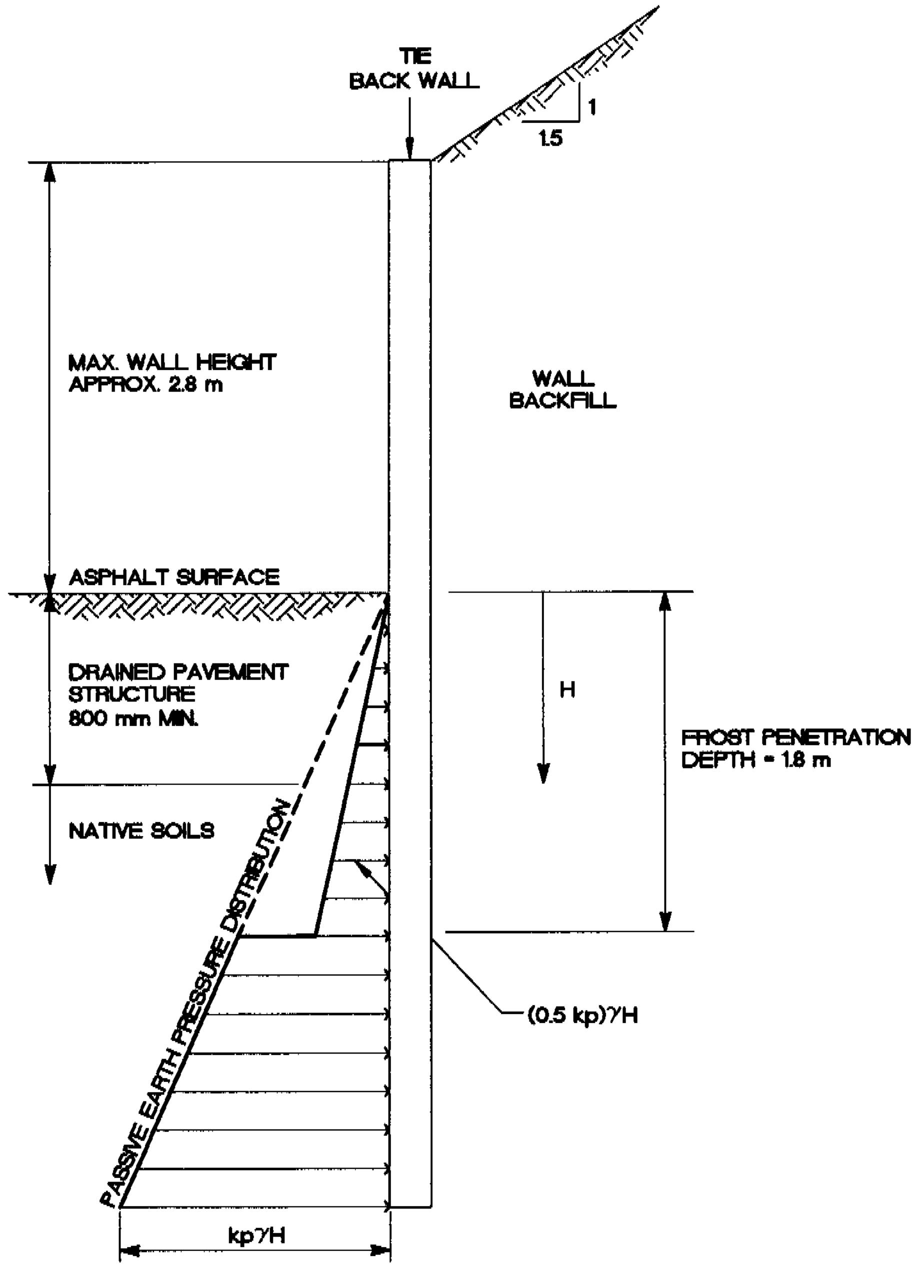


GWP 203-86-02
HWY 417 / MAITLAND AVE. BRIDGE REHABILITATION
RETAINING WALL BACKFILL

Scale: N.T.S.	Drawing No.: 11099-5	
Date: 98/11/04	Dwn. by: GBB	Appd.: PC

Jacques Whitford

11099-d6



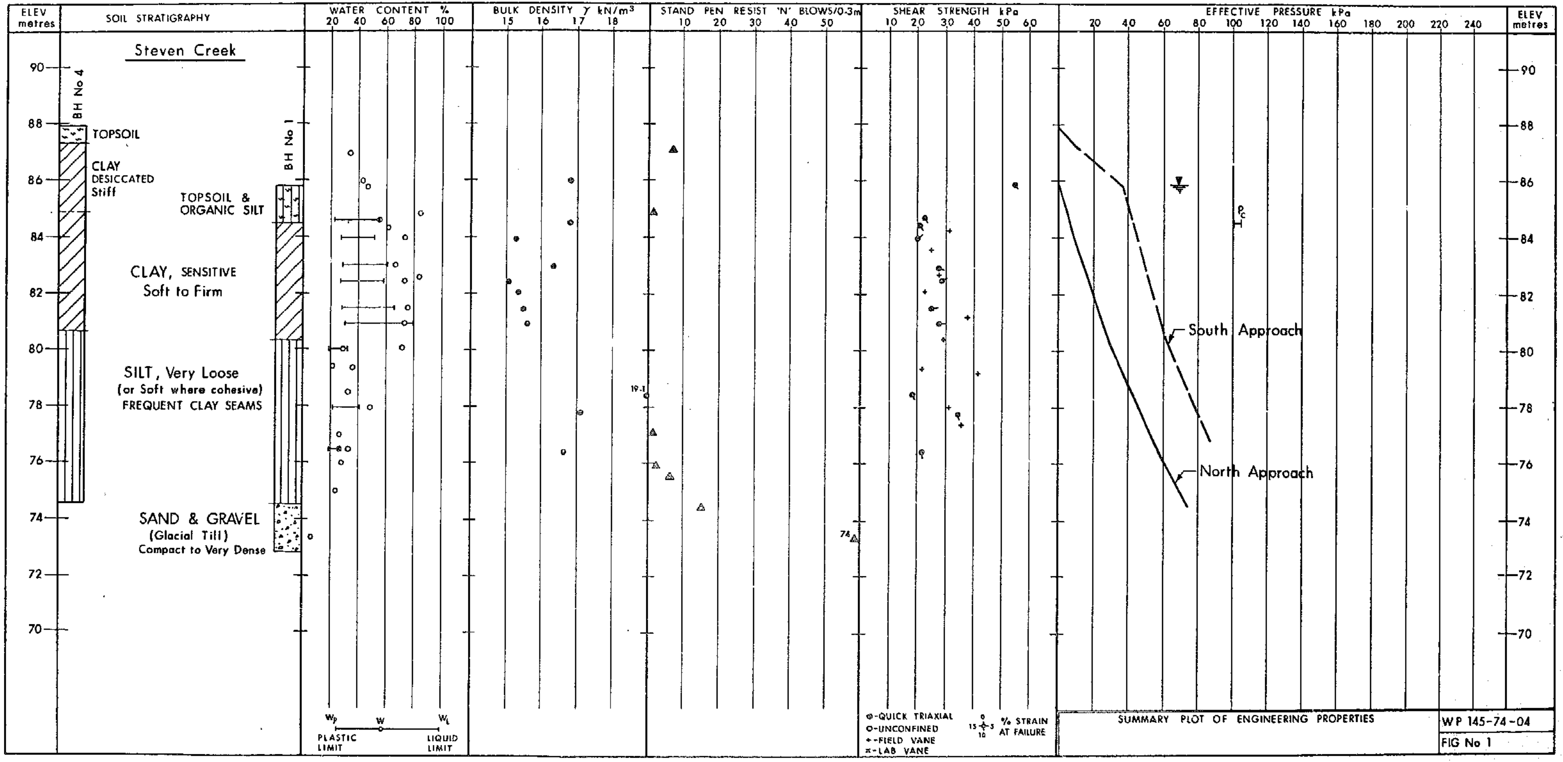
GWP 203-86-02
HWY 417 / MAITLAND AVE. BRIDGE REHABILITATION
PASSIVE EARTH PRESSURE DISTRIBUTION

Scale: N.T.S.	Drawing No.: 11099-6	
Date: 98/11/06	Dwn. by: GBB	Appd.: PC

Jacques Whitford

FF-A-25

SUMMARY PLOT OF ENGINEERING PROPERTIES



SUMMARY PLOT OF ENGINEERING PROPERTIES

WP 145-74-04

FIG No 1