

FOUNDATION INVESTIGATION AND DESIGN REPORT  
FOR  
WIDENING OF PIKE CREEK BRIDGE  
G.W.P. 60-00-00, SITE 6-75  
HIGHWAY 401  
TOWN OF TECUMSEH, ONTARIO

Distribution:

4 cc: Ministry of Transportation  
1 cc: Foundation Investigation Report, Ministry of Transportation  
1 cc: PML Hamilton  
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PML Ref: 01TF073B  
Geocres No. 40J2-55

November 2002

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## FOUNDATION INVESTIGATION REPORT

for

Widening of Pike Creek Bridge

G.W.P. 60-00-00, Site 6-75

Highway 401

Town of Tecumseh, Ontario

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### **INTRODUCTION**

This report summarizes the results of the foundation investigation carried out for the proposed widening of the existing bridge at Pike Creek and Highway 401 in the Town of Tecumseh, Ontario. The investigation was conducted for the Southwestern Region Structural Section of the Ontario Ministry of Transportation.

Highway 401 passes over Pike Creek at approximate Station 18+792, Highway 401 chainage, in the Town of Tecumseh (Township of Sandwich South). The existing bridge consists of a single span structure with a width of about 30 m and span of 9 m.

The report pertains to the proposed bridge widening and approaches within about 20 m of the abutments.

### **SITE DESCRIPTION**

The site is situated at the intersection of the existing Highway 401 and Pike Creek. The bridge structure to be widened carries Highway 401 traffic over Pike Creek. At the location of the bridge, Highway 401 runs in the east-west direction.

The site is located in the Township of Sandwich South in Essex County (Southwestern Ontario), some 10 km east of Windsor along Highway 401. The surrounding lands are mainly level and used for a mix of residential and agricultural purposes.

The area is part of the Essex Clay Plain physiographic sub-region. It is essentially a till plain smoothed by deposits of lacustrine clay which settled in the depressions while the knolls were being lowered by wave action. In general, the soil in the sub-region consists of silty clays and/or clayey silts. The bedrock belonging to the Dundee Formation and anticipated at depths of 35 to 40 m is largely composed of Middle Devonian limestone, dolostone and shale.

### **INVESTIGATION PROCEDURES**

The field work was carried out during the period May 6 to 14, 2002 and comprised four boreholes advanced to depths of 6.6 to 14.2 m, as summarized in the following table, at the locations indicated on Drawing 1 (Appendix B):

Location	Borehole No.	Depth, m
West Abutment, North Side	75-1	8.1
West Abutment, South Side	75-2	14.2
East Abutment, North Side	75-3	14.2
East Abutment, South Side	75-4	6.6

The locations of and ground surface elevations at the boreholes were established in the field by Peto MacCallum Ltd. The following benchmark (BM) was used for vertical reference:

**BM VCP515:**    Top of round iron bar on  
                         south side of Highway 401  
                         east of Pike Creek 16.066  
                         RT 18+814.372  
                         Elevation 185.965 (geodetic)

The boreholes were advanced using continuous flight solid stem augers, powered by a track-mounted CME-75 Nodwell drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff.

Representative samples of the soil were recovered at frequent depth intervals using a conventional split spoon sampler during drilling. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. In situ vane shear and pocket penetrometer tests were also performed to further assess the shear strength of the cohesive soils.

The groundwater conditions in the boreholes were closely monitored during the course of the field work. Upon completion of drilling, the boreholes were backfilled with auger cuttings to the ground surface.

All of the recovered samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determinations. Atterberg Limits tests and grain size distribution analyses were carried out on selected samples, their results being presented in Figures 1 and 2 (Appendix A) and on the Record of Borehole sheets (Appendix B).

### **SUMMARIZED SUBSURFACE CONDITIONS**

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, boundary elevations, standard penetration and in situ vane shear/pocket penetrometer test results, groundwater observations, the results of laboratory Atterberg Limits tests, grain size distribution analyses and moisture content determinations. Samples submitted for laboratory testing are also shown on the borehole logs.

The borehole locations and stratigraphic profiles prepared from the borehole data are presented on Drawing 1.

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised a surficial fill underlain by silty clay till and discontinuous deposits of clayey/sandy silt till. The strata encountered are summarized below.

### Fill

Fill was present surficially in boreholes 75-2 to 75-4. The fill was 1.0 to 2.4 m thick and consisted of stiff silty clay. The moisture content of the fill was about 20%. It is noted that inclusions of topsoil were identified within the fill in borehole 75-3.

It is noteworthy that the boreholes were drilled at the toe of slope of the highway embankment fill. The embankment fill at this location is approximately 2 m high.

### Topsoil

Topsoil was identified directly beneath the fill in borehole 75-4. It was 200 mm thick and consisted of silty clay.

### Silty Clay Till

Cohesive silty clay till was encountered surficially (elevation 184.8) in borehole 75-1 and at depths of 1.0 to 2.4 m (elevation 183.0 to 184.0) in the remaining boreholes. The consistency of the clay till was hard to firm, typically very stiff to stiff. Standard penetration test 'N' values ranged from 16 to 42 in the upper 4 m of all the boreholes, decreasing to a range of 7 to 19 beneath. The results of vane shear testing carried out in the lower part of this stratum indicate that the undisturbed and remolded shear strength values are about 80 and 45 kPa respectively (soil sensitivity is about 1.8). A number of pocket penetrometer tests conducted within the unit gave the values of undrained shear strength varying between 20 and 85 kPa, generally decreasing with depth. (Values less than about 30 kPa were likely obtained from testing of disturbed samples.)

The moisture content of the clay till ranged from 15 to 22%. The results of the Atterberg Limits tests are presented in Figure 1 (Appendix A). The clay till plots as a clay of low plasticity, with a plastic limit of 16 and liquid limits of 33 and 34. The results of particle size distribution analyses conducted on the clay till are presented in Figure 2 (Appendix A).

The clay till had a thickness of 9.3 m in borehole 75-3 and was not penetrated upon termination of the remaining boreholes at depths of 6.6 to 14.2 m (elevation 170.8 to 178.3).

#### Clayey Silt Till

Clayey silt till (judged to slight plasticity) was revealed below the silty clay till at 11.7 m depth (elevation 173.7) in borehole 75-3. This unit was 1.5 m thick and very stiff in relative density/consistency (N-value of 17). The moisture content of the unit was 11%.

#### Sandy Silt Till

Underlying the clayey silt till at 13.2 m depth (elevation 172.2) in borehole 75-3 was cohesionless sandy silt till. This deposit was compact (N-value of 10) and had a moisture content of 12%. The sandy silt till was not penetrated upon termination of the borehole at 14.2 m depth (elevation 171.2).

#### Groundwater

No water was observed in any of the boreholes during the course of the field work. It is noted, however, that the water level in Pike Creek measured in January 2001 was at elevation 182.1.

Based on visual examination of the samples retrieved during drilling and water level observations/measurements during field investigations conducted for other structures throughout the study corridor, the stabilized groundwater level at this site is expected to be near the water level in the adjacent creek, 182.1 at the time of drilling.

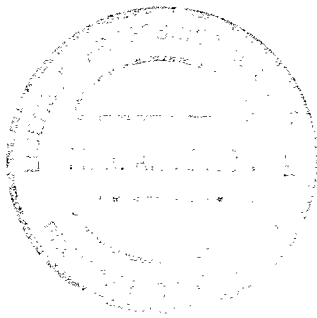
Groundwater levels may fluctuate subject to seasonal variations and precipitation patterns. "Flood conditions" may occur during construction resulting in surface water flow into the excavations.



**CLOSURE**

The field work was carried out under the supervision of Mr. M. Rapsey and direction of Mr. M.R. Anderson, M.Eng., P.Eng., Senior Foundation Engineer. The equipment was supplied by All-Terrain Drilling Limited.

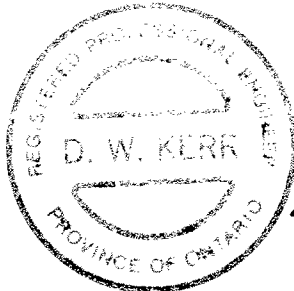
The report was prepared by Mr. G.O. Degil, Ph.D., Senior Project Supervisor, and Mr. M.R. Anderson, M.Eng., P.Eng., Senior Foundation Engineer. It was reviewed by Mr. D.W. Kerr, M.Eng., P.Eng., Chief Foundation Engineer. Mr. B.R. Gray, M.Eng., P.Eng., President, carried out an independent review of the report.



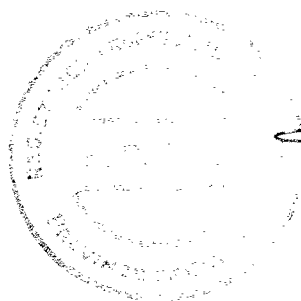
Yours very truly

**Peto MacCallum Ltd.**

Murray R. Anderson, M.Eng., P.Eng.  
Senior Foundation Engineer



Dennis W. Kerr, M.Eng., P.Eng.  
Chief Foundation Engineer



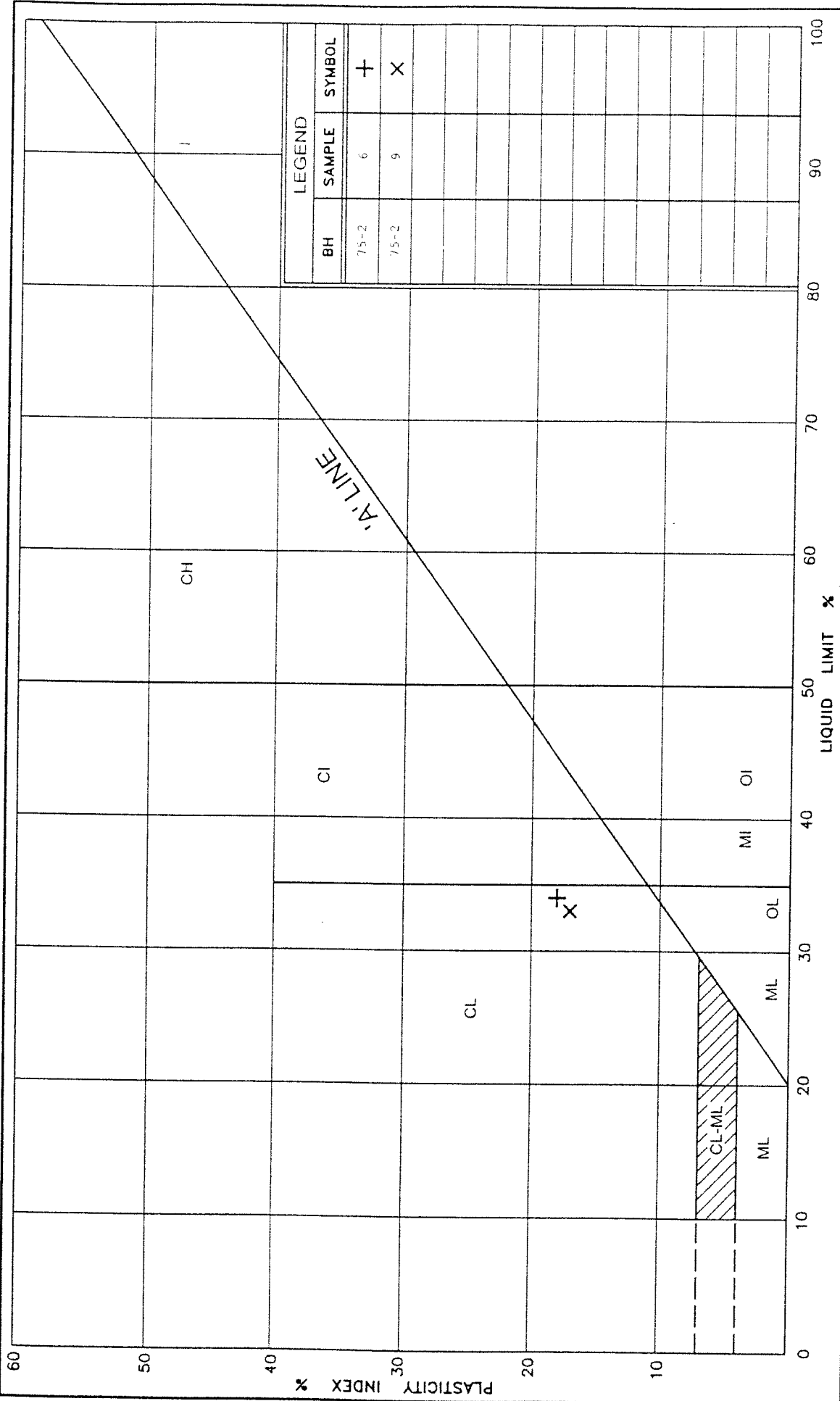
Brian R. Gray, M.Eng., P.Eng.  
President

GD:lad

## **APPENDIX A**

Figure 1 – Plasticity Chart

Figure 2 – Grain Size Distribution Chart



LEGEND		
BH	SAMPLE	SYMBOL
75-2	6	+
75-2	9	X



# PLASTICITY CHART

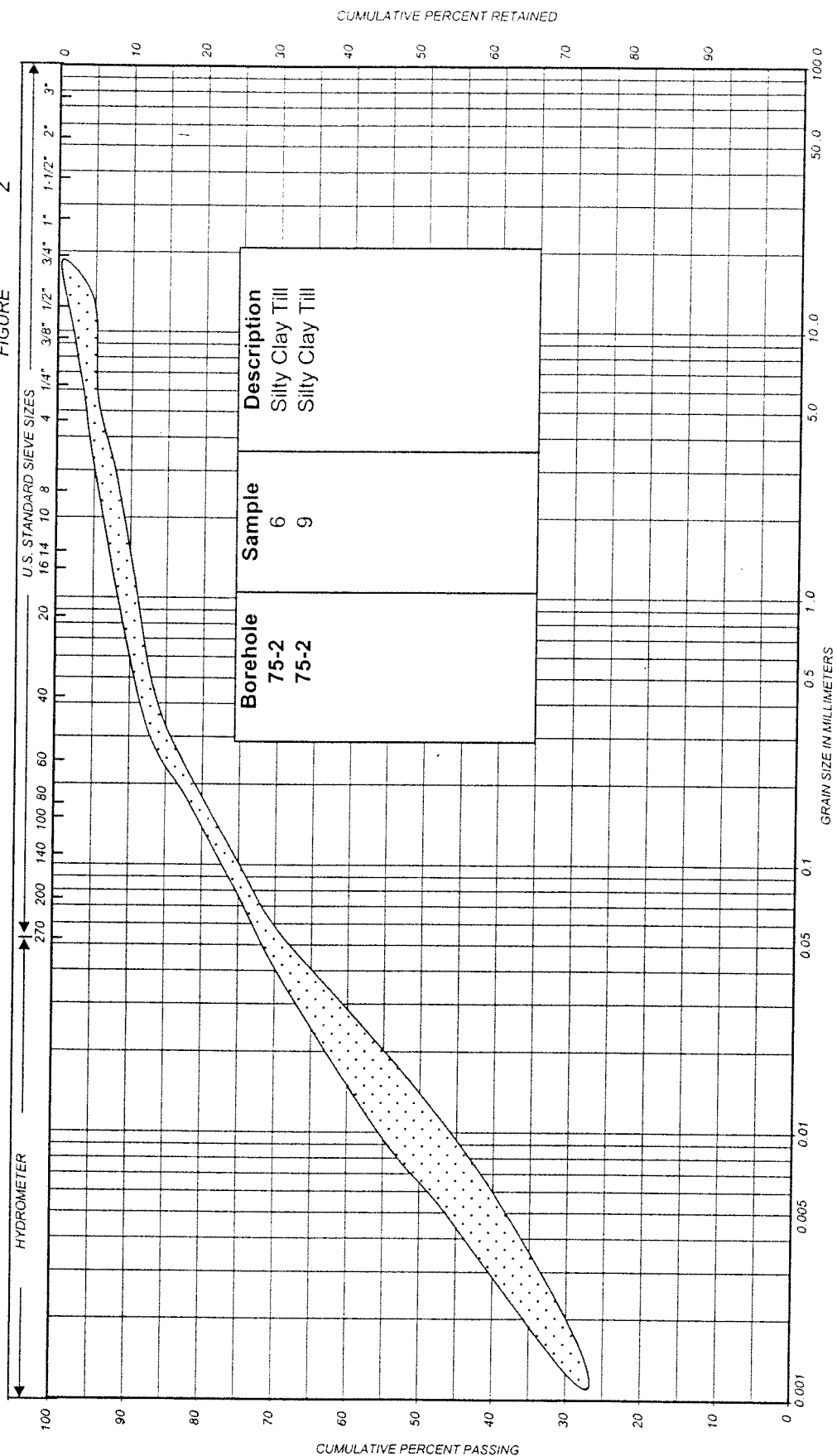
SILTY CLAY, with sand, trace of gravel (CL)

FIG No 1

HIGHWAY 401

G.W.P. 60-00-00, Site 6-75

PML REF.	01TF073B
G.W.P.	60-00-00
FIGURE	2

[illegible]REMARKS SILTY CLAY TILL, with sand, trace of gravel (CL)

## **APPENDIX B**

Record of Borehole Sheets

Drawing 1

## LIST OF ABBREVIATIONS

### PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N'. - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 0.3 m INTO THE SUBSOIL. DRIVEN BY MEANS OF A 63.5 kg HAMMER FALLING FREELY A DISTANCE OF 0.76 m.

DYNAMIC PENETRATION RESISTANCE. - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 51 mm, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 0.3 m INTO THE SUBSOIL. THE DRIVING ENERGY BEING 475 J PER BLOW.

### DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS:

<u>CONSISTENCY</u>	<u>'N' BLOWS/0.3 m</u>	<u>c kPa</u>	<u>DENSENESS</u>	<u>'N' BLOWS/0.3 m</u>
VERY SOFT	0 – 2	0 – 12	VERY LOOSE	0 – 4
SOFT	2 – 4	12 – 25	LOOSE	4 – 10
FIRM	4 – 8	25 – 50	COMPACT	10 – 30
STIFF	8 – 15	50 – 100	DENSE	30 – 50
VERY STIFF	15 – 30	100 – 200	VERY DENSE	> 50
HARD	> 30	> 200		
W.T.P.L. WETTER THAN PLASTIC LIMIT			D.T.P.L. DRIER THAN PLASTIC LIMIT	
A.P.L. ABOUT PLASTIC LIMIT				

### TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
P.H.	SAMPLE ADVANCED HYDRAULICALLY		
P.M.	SAMPLE ADVANCED MANUALLY		

### SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL		

▲, Δ - UNDISTURBED AND REMOULDED SHEAR STRENGTH DETERMINED FROM IN SITU VANE TEST.

■ - UNDRAINED SHEAR STRENGTH DETERMINED FROM POCKET PENETROMETER TEST.

RECORD OF BOREHOLE No 75-1

1 of 1 METRIC

G.W.P. 80-04-00 LOCATION Grands, 1.428 055 N; 212 360 E. ORIGINATED BY MR  
DIST 32 HWY 101 BOREHOLE TYPE Rotary Flight Solid Stem Auger COMPILED BY TD  
DATUM Geodetic DATE May 14, 2001 CHECKED BY MPA

SOIL PROFILE --			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	T <sub>N</sub> VALUES			20 40 60 80 100	20 40 60 80 100					
194.40	Ground Level													
193.99	Silty clay, with sand, trace of gravel Very Stiff		1	SS	16		184							
	Brown (Till)		2	SS	22		183							
			3	SS	25		182							
			4	SS	25		181							
			5	SS	19		180							
	Stiff		6	SS	13		179							
	Grey		7	SS	9		178							
176.70	End of Borehole		8	SS	10		177							
8.10	Borehole dry on completion of drilling													

RECORD OF BOREHOLE No 75-2

1 of 1 METRIC

G.W.P. 60-00-10 LOCATION 100-118.1 ATR 014 N; 271 560 S. ORIGINATED BY MR  
DIST 22 HWY 401 BOREHOLE TYPE Continuous Flight Drill Stem Auger COMPILED BY JS  
DATUM Geodetic DATE May 16, 2002 CHECKED BY MRS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								20 40 60 80 100										
								20 40 60 80 100										
185.03	Ground Level																	
0.90	Silty clay, trace of sand Stiff Brown Mottled (Fill)																	
184.03			1	SS	14		184											
1.00	Silty clay, with sand, trace of gravel Very Stiff to Hard Brown (Fill)		2	SS	32		183											
			3	SS	26		182											
			4	SS	26		181											
			5	SS	26		180											
	Stiff to Firm Grey		6	SS	11		179							1 CL 45 30				
			7	SS	11		178											
			8	SS	9		177											
			9	SS	8		176							6 CL 37 36				
			10	SS	7		175											
			11	SS	10		174											
			12	SS	9		173											
170.83	End of Borehole Borehole dry on completion of drilling ■ Penetrometer Test						172											
14.20							171											



RECORD OF BOREHOLE No 75-3

1 of 1 METRIC

G.W.P. 60-20-00 LOCATION (Coordinates: 1 673 101 W 323 124 E) ORIGINATED BY MS  
DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Auger System Augers COMPILED BY GT  
DATUM Geodetic DATE May 11, 2002 CHECKED BY MPA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATPILOT	NUMBER	TYPE	TEST VALUES			20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W	W <sub>L</sub>		
185.10	Ground Level													
0.00	Silty clay, trace to some sand, with occ. inclusions of topsoil													
	Stiff													
	Brown (Fill)		1	SS	9		145							
			2	SS	8		144							
182.95														
0.45	Silty clay, with sand, trace of gravel		3	SS	10		182							
	Very Stiff to Hard													
	Brown (Till)		4	SS	12		182							
	Grey		5	SS	15		181							
			6	SS	13		180							
	Stiff													
			7	SS	11		179							
			8	SS	9		178							
				FV			177							
			9	SS	10		176							
				FV			175							
			10	SS	11		174							
173.70														
11.70	Clayey silt, some sand, trace of gravel													
	Very Stiff													
	Grey (Till)		11	SS	17		172							
172.20														
13.20	Sandy silt, trace of clay and gravel													
	Compact													
	Grey (Till)		12	SS	10		172							
171.20														
14.20	End of Borehole													
	Borehole dry on completion of drilling													
	Penetrometer Test													

ON\_MOT 01TF073B.GPJ ON\_MOT.GDT 11/19/2002 11:56:55 AM

+7, X5: Numbers refer to Sensitivity 20 15 10 5 1 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 75-4

1 of 1 METRIC

G.W.P. 60-00-00 LOCATION Co-ords. 4 678 912 N; 272 673 E. ORIGINATED BY MR  
DIST 32 HWY 101 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY GD  
DATUM Geodetic DATE May 07, 2002 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
184.45	Ground Level											
0.00	Silty clay, trace of sand Stiff Brown (Fill)		1	SS	9		184					
183.45												
183.00	Silty clay Dark Brown Topsoil		2	SS	20		183					
1.60	Silty clay, with sand, trace of gravel Very Stiff to Hard Brown (Till)		3	SS	34		182					
			4	SS	20		181					
	Very Stiff to Stiff Grey		5	SS	16		180					
			6	SS	12		179					
178.30			7	SS	11							
6.55	End of Borehole Borehole dry on completion of drilling  ■ Penetrometer Test											

# METRIC

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

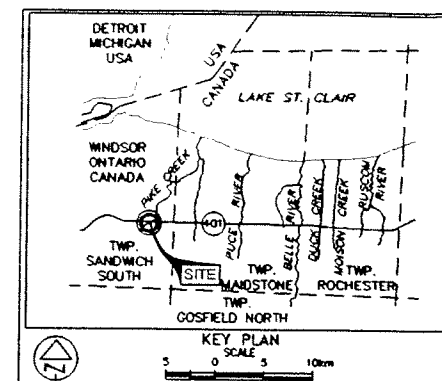
CONT No  
GWP No 60-00-00

HIGHWAY 401  
PROPOSED BRIDGE WIDENING AT PIKE CREEK  
BOREHOLE LOCATIONS & SOIL STRATA



SHEET

**Peto MacCallum Ltd**  
CONSULTING ENGINEERS



## LEGEND

- Borehole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Borehole & 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 May 2002
- Head
- ARTESIAN WATER
- Encountered

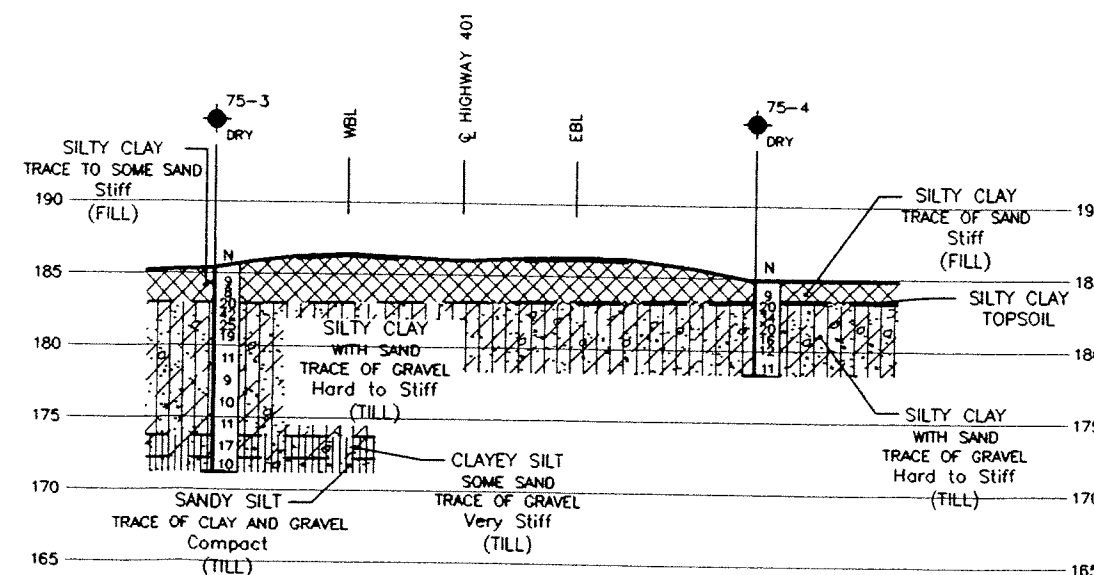
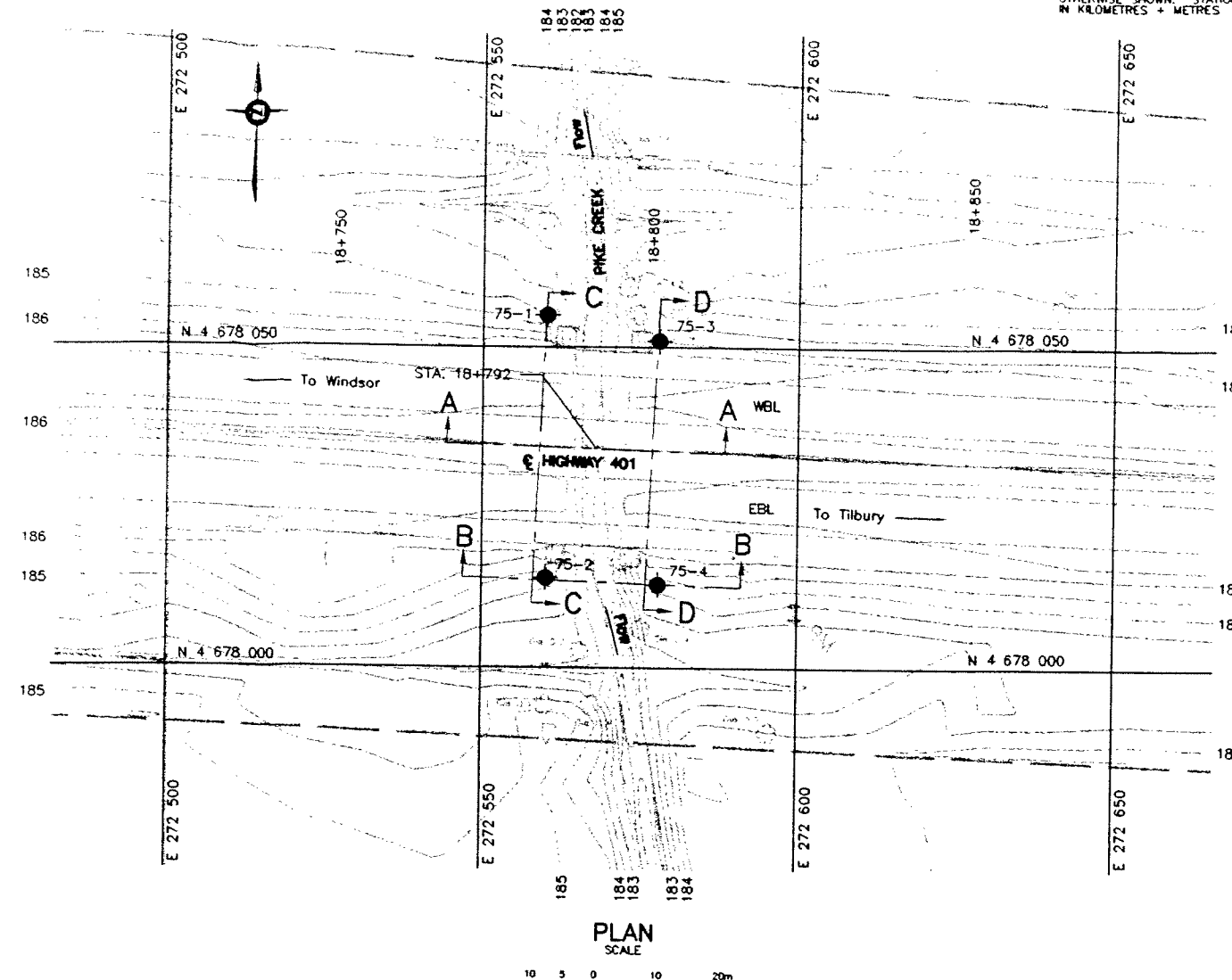
BH No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
75-1	184.80	4 678 055	272 560
75-2	185.03	4 678 014	272 560
75-3	185.40	4 678 051	272 578
75-4	184.85	4 678 013	272 578

## NOTE

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

DATE	BY	DESCRIPTION

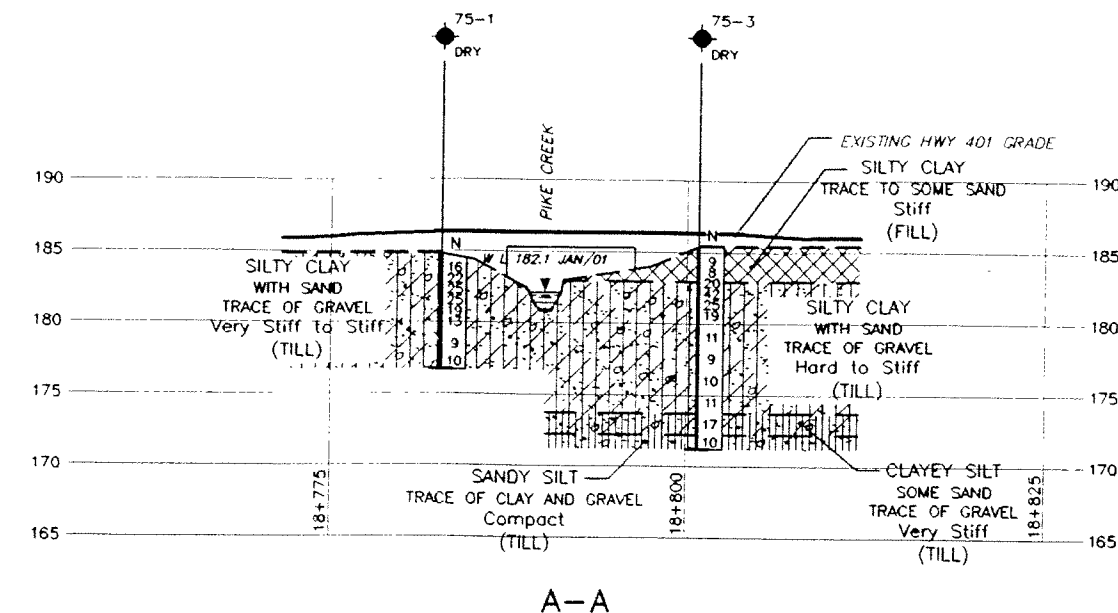
REF No Survey Plan 2001 Site # 8-75, entitled Proposed Crossing at Pike Creek and King's Highway 401. Prepared by Marshall Mocklin Monahan and



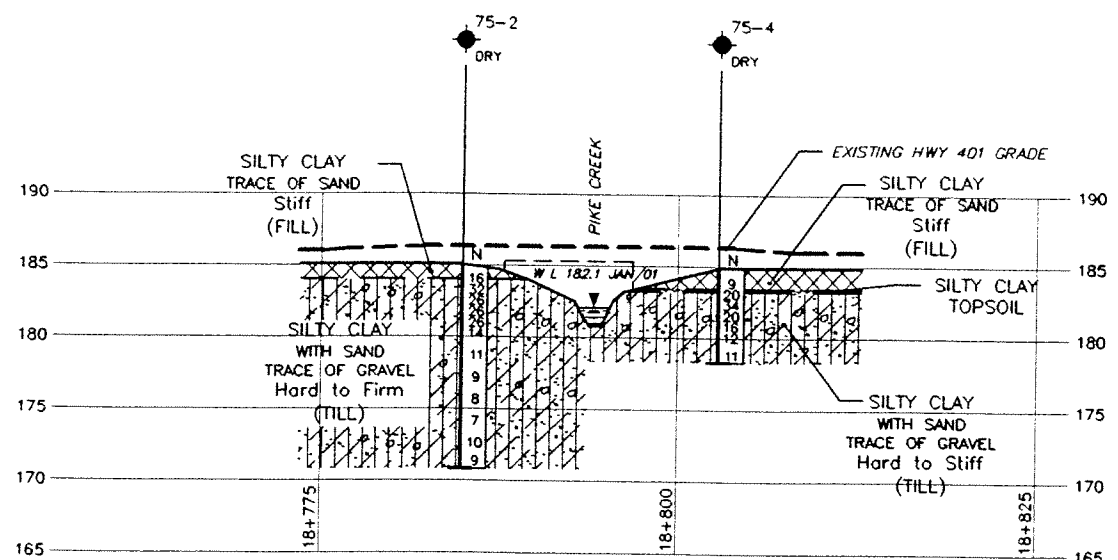
D-D

## SECTIONS

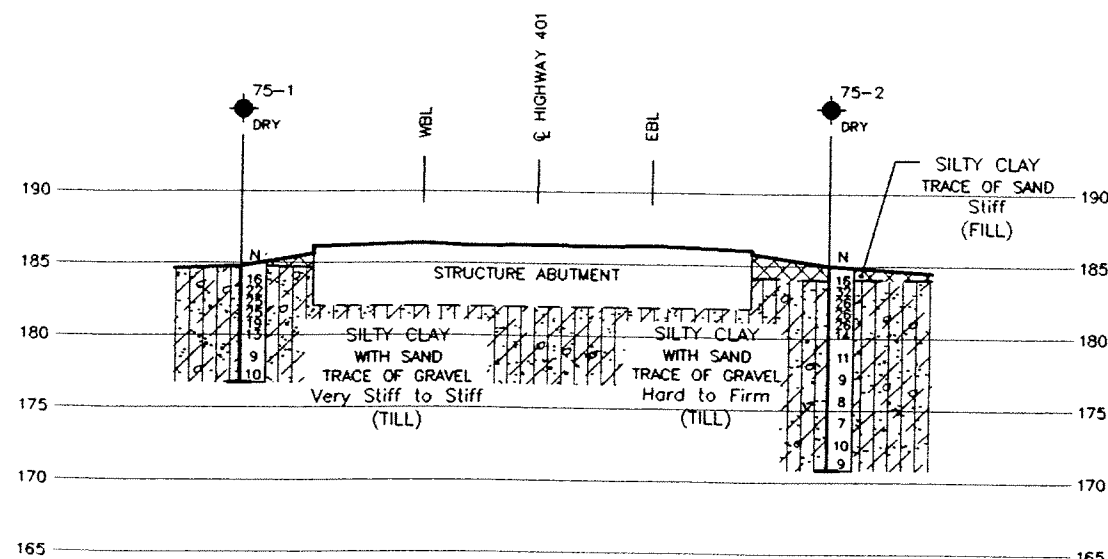
SCALE  
0 2.5 5 10m



A-A



B-B



C-C

## NOTE

SECTIONS ARE PROVIDED SOLELY FOR ILLUSTRATIVE PURPOSES. REFER TO RECORD OF BOREHOLES FOR DETAILED DESCRIPTION OF SUBSURFACE CONDITIONS, IN-SITU TEST DATA AND LABORATORY TEST RESULTS.

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November 2002

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**APPENDICES**

<b>FIGURE 1</b>	<b>-</b>	<b>LATERAL EARTH PRESSURE DISTRIBUTION: SINGLY-BRACED CUTS IN COHESIVE SOILS</b>
<b>FIGURE 2</b>	<b>-</b>	<b>LATERAL EARTH PRESSURE DISTRIBUTION: MULTI-BRACED CUTS IN COHESIVE SOILS</b>
<b>FIGURE 3</b>	<b>-</b>	<b>GENERAL RECOMMENDATIONS REGARDING UNDERPINNING OF FOUNDATIONS/UTILITIES LOCATED CLOSE TO EXCAVATION</b>

## FOUNDATION DESIGN REPORT

for

Widening of Pike Creek Bridge

G.W.P. 60-00-00, Site 6-75

Highway 401

Town of Tecumseh, Ontario

---

### INTRODUCTION

This report provides geotechnical comments and recommendations regarding design and construction of foundations, abutments and approaches for the proposed widening of the existing bridge at Pike Creek and Highway 401 in the Town of Tecumseh, Ontario. The investigation was conducted for the Southwestern Region Structural Section of the Ontario Ministry of Transportation.

The existing bridge consists of a single span structure with a width of about 30 m and span of 9 m (ref. drawing No. 401-SAN-E-01 'Proposed Crossing at Pike Creek. King's Highway 401 dated April 2001). Highway 401 will pass over Pike Creek at approximate Station 18+792, Highway 401 chainage.

Road grade on Highway 401 at the bridge location is near elevation 186.5 (interpolated from existing grade shown on the drawing referred to above). The existing approach embankments to the bridge comprise fill of approximately 2 m in height. It is understood that the existing bridge is supported on shallow spread footings.

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised a surficial fill underlain by silty clay till and discontinuous deposits of clayey/sandy silt till. The clay till exhibits an upper very stiff to hard crust and becomes very stiff to firm below a depth of about 4 m.

## FOUNDATIONS

### Spread Footings

Supporting the bridge widenings on conventional spread footings founded in the native soil is considered to be feasible at this site. The new footings should be founded at the same level as the existing footings.

Details concerning design of the foundation supporting the existing bridge were not provided. Based on the river bed elevation shown on the drawing provided and the engineering properties of the soil revealed in the boreholes drilled at the site for this study, it is assumed that the footings are founded near elevation 182.0.

For a 2.5 m wide strip footing constructed on the very stiff to hard clay till, the following values of bearing resistance at ultimate and serviceability limit states (ULS and SLS) are recommended at the inferred design founding level:

Factored Bearing Resistance at ULS = 375 kPa

Bearing Resistance at SLS = 250 kPa

Use of spread footings constructed on structural fill placed in the approach embankments does not appear to be suitable at this site since the embankment fill is about 3 m thick and a minimum 3.5 m thickness of engineered fill would be required to optimize the bearing resistance values at ULS and SLS.

The recommended resistance at SLS allows for 25 mm of total settlement; differential settlement is expected to be less than 75% of this value. A footing embedment depth of 1.2 m was assumed for computation of the ULS resistance.

Sliding would be resisted in part by the friction force developed between the underside of footing and the native clay till or granular fill. Unfactored friction factors of 0.35 and 0.45 are recommended for footings on clay till and granular fill, respectively.

All footings subject to frost action should be provided with the normal 1.2 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

Prior to placement of structural concrete, all foundation excavations should be examined by qualified geotechnical personnel to verify the competency of the founding surface.

### Deep Foundations

Supporting the structure widenings on deep foundations is not recommended considering the potential for incompatible settlement behaviour between the new and existing portions of the bridge.



## ABUTMENT WALLS

The abutment walls should be designed to resist the unbalanced horizontal earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure,  $p$ , may be computed using the equivalent fluid pressures presented in Section 6-7.4 of the Ontario Highway Bridge Design Code (OHBDC, 3rd Edition, 1991) or employing the following equation, assuming a triangular pressure distribution:

$$p = K (\gamma h + q)$$

where

$K$  = lateral earth pressure coefficient

$\gamma$  = unit weight of free-draining granular material ( $\text{kN/m}^3$ )

$h$  = depth below final grade (m)

$q$  = surcharge load (kPa) if present

Free-draining granular material should be used as backfill behind the wall. The following parameters are recommended for design:

	<u>Granular "A"</u>	<u>Granular "B"</u>
Angle of Internal Friction, degrees	35	32
Unit weight, $\text{kN/m}^3$	22.8	21.2
Coefficient of Active Earth Pressure $K_a$	0.27	0.31
Coefficient of Earth Pressure At Rest $K_o$	0.43	0.47
Coefficient of Earth Pressure At Rest $K_o$	3.69	3.25

The coefficient of earth pressure at rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained structures.

A weeping tile system and/or weep holes should be installed to minimise the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost-free outlet.

A retained soil system (RSS) could also be employed. The founding material is expected to comprise granular engineered fill or clay till. The following parameters should be used in design of the system foundation:

	<u>Granular "A"</u>	<u>Granular "B"</u>	<u>Silty Clay</u>
Friction Angle, degrees	35	32	0
Cohesion, kPa	0	0	125
Unit weight, kN/m <sup>3</sup>	22.8	21.2	20.4

The bearing resistance values previously indicated for abutment footing design should be employed for design of the RSS wall.

The supplier should be responsible for design of the RSS (backfill, reinforcement, internal and external stability) and provide drawings to show pertinent information such as location, length, height, elevations, performance level, appearance etc.

### **APPROACH EMBANKMENTS**

Backfilling adjacent to the abutments should be carried out in conformance with Ontario Provincial Standards specifications for granular backfill (OPSD 3501.00).

The embankments should be constructed with earth fill or granular material in accordance with OPSD 200.010, 202.010 and 208.010. Embankment slopes inclined no steeper than 2 horizontal to 1 vertical should be stable. Since the embankment fill height is less than 8 m, construction of a mid-height berm for erosion control and slope maintenance purposes should not be necessary.

Consolidation settlements due to placing fill on the inorganic native soil are expected to be less than 10 mm. No bearing capacity problems are anticipated. Any topsoil and other deleterious material should be stripped prior to placement of the approach fill.

### **EXCAVATION AND GROUNDWATER CONTROL**

Excavation for construction of footings is expected to extend through the embankment fill and topsoil layers. Based on the inferred design founding elevation of 182.0, the depth of excavation will vary from about 3 m beyond the toe of the embankment fill to as much as 4.5 m at the top of the fill. Cognizant of the fill present at the site, the materials are classified as Type 3 soils according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Temporary cut slopes inclined at 45° to the horizontal should generally be stable. Flatter side slopes may be required if excessively soft/wet materials or concentrated seepage zones are encountered locally.

Shoring may be required to support the walls of the excavation and adjacent traffic lanes during construction of the foundations and approach embankments if traffic is to be maintained during construction.

The magnitude and distribution of the lateral earth pressures acting on a braced excavation wall is dependent upon the support system used, the number of supports, the allowable movements and the construction sequence. The recommended design earth pressure distribution for singly and multi-braced walls, for the conditions that exist at the site, are presented in Figures 1 and 2 respectively. Recommendations concerning design and construction of the braced excavation support systems are provided in the figures.

The magnitude and distribution of the lateral earth pressures acting on a braced excavation wall is dependent upon the support system used, the number of supports, the allowable movements and the construction sequence. The recommended design earth pressure distribution for singly and multi-braced walls, for the conditions that exist at the site, are presented in Figures 1 and 2 respectively. Recommendations concerning design and construction of the braced excavation support systems are provided in the figures.

Use of steel sheeting is recommended due to the proximity of Pike Creek and the potential for "flood conditions" during construction.

A soldier pile and lagging system could be considered. Provided the spacing between soldier piles is at least five pile diameters, the unfactored lateral passive resistance developed on the face of the soldier pile below the base of the excavation may be taken as the passive earth pressure developed over an equivalent wall area of width three times the pile diameter and depth of six times the pile diameter. A passive earth pressure coefficient  $K_p$  of 3.0 is recommended for this computation.

Additional lateral resistance could be provided by installing tiebacks anchored in the very stiff to hard clay till. The factored pull-out resistance at ULS of soil anchors in the clay till may be computed as follows:

$$R = 0.4c_u A_s L_s$$

where

$c_u$  = average undrained shear strength over the anchor length

= 125 kPa for very stiff to hard clay till

$A_s$  = effective unit surface area of the anchor ( $m^2$ )

$L_s$  = effective embedment length of the anchor (m)

The ground surface adjacent to the excavation is expected to experience some inward movement and vertical settlement. The magnitude of movements adjacent to a braced cut can be limited by selection of an appropriate lateral earth pressure coefficient (see Figures 1 and 2) provided good quality workmanship and construction practice is employed. The anticipated magnitude of movements is as follows:

<u>Movement (% of Excavation Depth)</u>	
Lateral Movement	
Braced Excavation	0.2
Anchored Wall	0.1
Vertical Movement	0.05

Construction procedures should be specifically suited to limit any consequent settlement of the pavement subgrade behind the excavation face.

Foundations of heavily loaded/settlement sensitive structures and/or utilities located within close proximity to the excavation may require underpinning to preserve the integrity of these structures. Further comments and general recommendations in this regard are provided in Figure 3.

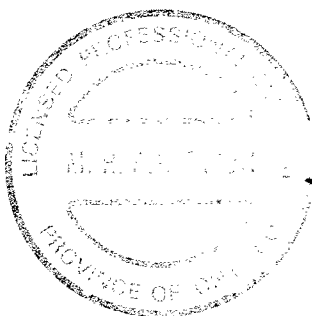
During the field investigation, no water was observed in any of the boreholes. It is anticipated that stabilized groundwater levels will be near the water level in adjacent Pike Creek (elevation 182.1 in January 2001). The stabilized groundwater level may vary in response to seasonal fluctuations and precipitation patterns. Further, "flood conditions" in Pike Creek may prevail during construction and surface water may flow into the excavations.

Considering the low permeability characteristics of the predominant clay till, groundwater seepage or surface water that enters excavation within the till should be readily handled by conventional sump pumping techniques. Extensive pumping would be required during flood periods. In this regard, if bracing of the excavation is required, use of steel sheeting to support the walls of the excavation is recommended.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.

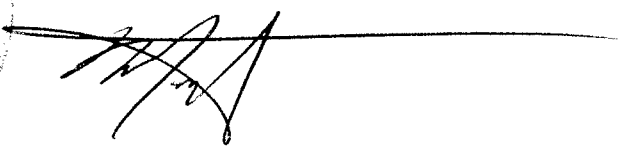
**CLOSURE**

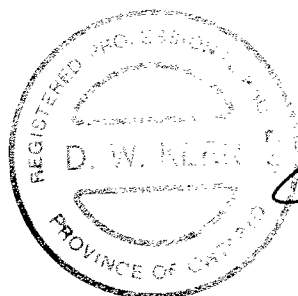
The report was prepared by Mr. G.O. Degil, Ph.D., Senior Project Supervisor, and Mr. M.R. Anderson, M.Eng., P.Eng., Senior Foundation Engineer. It was reviewed by Mr. D.W. Kerr, M.Eng., P.Eng., Chief Foundation Engineer. Mr. B.R. Gray, M.Eng., P.Eng., President, carried out an independent review of the report.




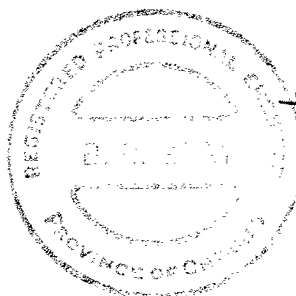
Yours very truly

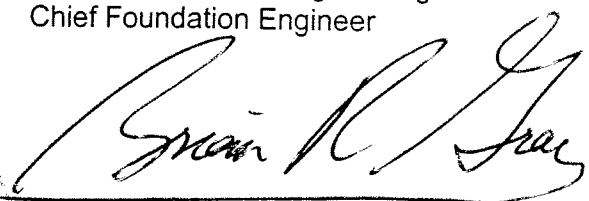
**Peto MacCallum Ltd.**

  
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President

GD:lad

## **APPENDICES**

FIGURE 1 – LATERAL EARTH PRESSURE DISTRIBUTION:  
SINGLY-BRACED CUTS IN COHESIVE SOILS

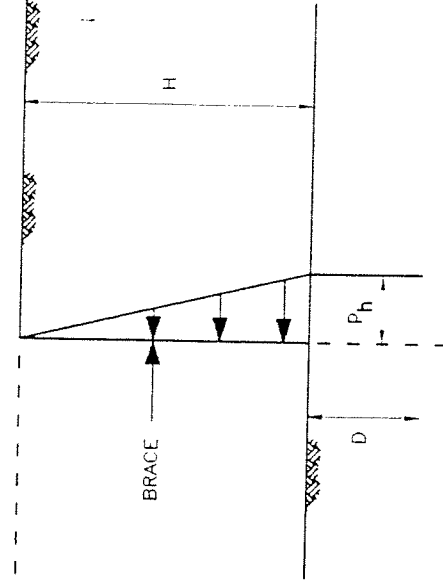
FIGURE 2 – LATERAL EARTH PRESSURE DISTRIBUTION:  
MULTI-BRACED CUTS IN COHESIVE SOILS

FIGURE 3 – GENERAL RECOMMENDATIONS REGARDING  
UNDERPINNING OF FOUNDATIONS/UTILITIES  
LOCATED CLOSE TO EXCAVATION

# NOTES

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system are dependent upon the permissible lateral/vertical movements adjacent to the excavation, the soil type, groundwater conditions, drainage provisions, temporary/permanent surcharge loads, the type of bracing system adopted, weather conditions, quality of workmanship and length of time the excavation will be supported. Hence, the recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system are established.
2. Stability of base of excavation must be confirmed when bracing system design, excavation geometry and surcharge loads are established.
3. Earth pressure diagram is applicable to maximum depth of cut of 12m (40 ft.).
4. Structural components of bracing system should be confirmed adequate for each level of excavation.
5. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.
6. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system are not included in earth pressure diagram.
7. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.
8. If settlement sensitive structures are located near the excavation, special measures should be undertaken to control settlements. A condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction.
9. Earth pressure diagram is applicable for relatively short construction periods. If excavation is to be open for long periods, monitoring of deformation is essential, the earth pressure diagram must be reviewed, and remedial works may be required.
10. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.
11. Bracing system should be regularly examined for signs of distress.
12. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.
13. This sheet should be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

## EARTH PRESSURE DIAGRAM



$$P_h = \text{design lateral earth pressure} \\ = K\gamma H$$

$$K = \text{lateral earth pressure coefficient}$$

$$\gamma = \text{unit weight of soil}$$

$$H = \text{depth of excavation}$$

$$D = \text{depth of embedment of soldier piles (if used)}$$

## RECOMMENDED DESIGN PARAMETERS

$$\gamma = 20.4 \text{ kN/m}^3$$

$$K = 0.35 \text{ (movement of retained soil acceptable)}$$

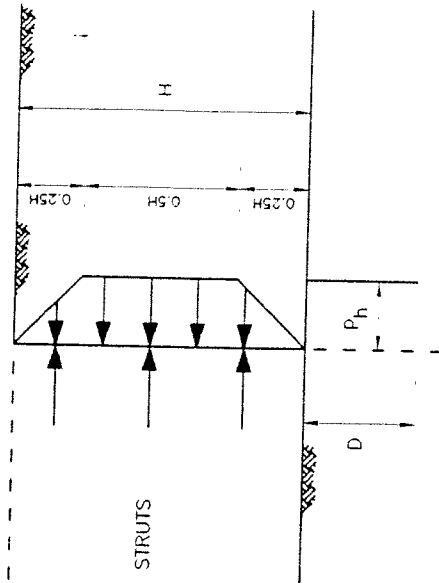
$$0.50 \text{ (movement of adjacent structures/facilities unacceptable)}$$



# NOTES

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system are dependent upon the permissible lateral/vertical movements adjacent to the excavation, the soil type, groundwater conditions, drainage provisions, temporary/permanent surcharge loads, the type of bracing system adopted, weather conditions, quality of workmanship and length of time the excavation will be supported. Hence, the recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system are established.
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## EARTH PRESSURE DIAGRAM



$P_h$  = design lateral earth pressure  
 $\gamma = 0.4 \gamma H$

where

$\gamma$  = unit weight of soil

$H$  = depth of excavation

$D$  = depth of embedment of soldier piles (if used).

## RECOMMENDED DESIGN PARAMETERS

$\gamma = 20.4 \text{ kN/m}^3$

# NOTES

1. The need to underpin existing footings/utilities is dependent upon soil type, proximity of the existing facility to the face of the excavation, loads imposed on the foundation and permissible movements.

## ZONE A:

Foundations of relatively heavy and/or settlement sensitive structures/utilities located in Zone A generally require underpinning.

## ZONE B:

Foundations of structures located within Zone B generally do not require underpinning. Consideration should be given to underpinning of settlement sensitive utilities or heavy foundation units located in this zone.

## ZONE C:

Utilities and foundations located within Zone C do not normally require underpinning.

Underpinning of foundations located in Zones A and B should extend at least into Zone C.

2. As an alternative to underpinning, it may be possible to control movement of existing utilities and foundations by supporting the face of the excavation with bracing/tiebacks or a rigid (caisson) wall. Horizontal and vertical earth pressures imposed on the excavation wall by non-underpinned foundations must be considered in the design of the support system.

3. A condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction to monitor any movement which may occur.

4. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.

5. This sheet is to be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

