Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
ST. JOACHIM ROAD OVERPASS
G.W.P. 60-00-00, SITE 6-87
HIGHWAY 401
TOWN OF LAKESHORE, ONTARIO

Distribution:

4 cc: Ministry of Transportation
1 cc: Foundation Investigation Report, Ministry of Transportation
1 cc: PML Hamilton
1 cc: PML Toronto

PML Ref: 01TF073G
Geocres No. 40J2-53

November 2002

PetoMacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION REPORT
FOR
ST. JOACHIM ROAD OVERPASS
G.W.P. 60-00-00, SITE 6-87
HIGHWAY 401
TOWN OF LAKESHORE, ONTARIO

Distribution:

4 cc: Ministry of Transportation
1 cc: Foundation Investigation Report, Ministry of Transportation
1 cc: PML Hamilton
1 cc: PML Toronto

PML Ref: 01TF073G
Geocres No. 40J2-53

November 2002

Peto MacCallum Ltd.
CONSULTING ENGINEERS

TABLE OF CONTENTS

FOUNDATION INVESTIGATION REPORT
for
St. Joachim Road Overpass
G.W.P. 60-00-00, Site 6-87
Highway 401
Town of Lakeshore, Ontario

INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed replacement of the existing overpass structure at St. Joachim Road and Highway 401 in the Town of Lakeshore, Ontario. The investigation was conducted for the Southwestern Region Structural Section of the Ontario Ministry of Transportation.

Highway 401 will pass over St. Joachim Road at approximate Station 16+725, Highway 401 chainage, in the Town of Lakeshore (Township of Rochester).

The report pertains to the proposed overpass structure and approaches within about 20 m of the abutments.

SITE DESCRIPTION

The site is situated at the intersection of the existing Highway 401 and St. Joachim Road. The proposed structure will carry Highway 401 traffic over St. Joachim Road. At the location of the structure, Highway 401 runs in the east-west direction. The existing approaches comprise fill embankments with a height of approximately 6 m.

The site is located in the Town of Lakeshore in Essex County (Southwestern Ontario), some 30 km east of Windsor along Highway 401. The surrounding lands are mainly level and used for a mix of agricultural and residential 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 a depth of about 35 m is largely composed of Middle Devonian limestone, dolostone and shale.

INVESTIGATION PROCEDURES

The field work was carried out during the period of January 17 to April 1, 2002 and comprised six boreholes advanced to depths of 5.0 to 33.4 m, as summarized in the following table, at the locations indicated on Drawing 1 (Appendix B).

Location	Borehole No.	Depth (m)		
		Auger	Rock Core [1]	Total
West Approach, South Side	87-1	5.0	-	5.0
West Abutment, South Side	87-2	9.6	-	9.6
West Abutment, North Side	87-3	33.4	3.0	36.4
East Abutment, South Side	87-4	32.5	2.9	35.4
East Abutment, North Side	87-5	9.6	-	9.6
East Approach, North Side	87-6	5.0	-	5.0

(1) NQ diamond rock coring equipment.

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:

MTCBM 738556: Tablet in east face of west abutment
Elevation 183.481 (geodetic)

The boreholes were advanced using continuous flight solid and hollow stem augers as well as mud rotary methods, powered by track-mounted CME-75 Nodwell and truck-mounted CME-75 drill rigs, supplied and operated by specialist drilling contractors, 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 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. A piezometer was installed in borehole 87-2 to monitor groundwater levels.

The deep boreholes were sealed with cement-bentonite grout upon completion of drilling and coring. The remaining 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, rock core descriptions, 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 Drawings 1 and 2.

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised a surficial topsoil underlain by clay till. Limestone bedrock was contacted below the clay till at depths of 32.5 and 33.4 m (elevation 150.3 and 149.7). The strata encountered are summarized below.

Fill

Fill was revealed in borehole 87-3. It consisted of a surficial 750 mm thick layer of fine to coarse sand and gravel over a 950 mm thick layer of silty clay. The latter had a moisture content of 22 to 25%.

It is noteworthy that the boreholes were not drilled through the existing embankment fill. The highway embankment fill at this location is about 6 m high (Plan 31ROC-E, May 2001, prepared by J.D. Barnes Limited).

Topsoil

Topsoil was present surficially in all the boreholes except borehole 87-3. It consisted of silty clay and was 100 to 200 mm thick.

Clay Till

Cohesive silty clay till was encountered directly beneath the topsoil (fill in borehole 87-3) at elevation 181.4 to 183.1. The consistency of the clay till was stiff to hard in the upper 10 to 14 m thick portion of the unit, becoming generally firm to stiff below this depth. The results of vane shear testing carried out in this stratum between approximate depths of 8.5 and 16.0 m indicate that the undisturbed and remolded shear strength values are 65 to 170 kPa and 40 to 120 kPa respectively (soil sensitivity is about 1.5). A number of penetrometer tests conducted within the unit gave values of undrained strength varying broadly between 35 and 210 kPa, typically decreasing with depth from about 100 to 40 kPa.

The moisture content of the clay till ranged from 14 to 24% in the upper portion of the unit, reaching 19 to 31% at depth. The results of the Atterberg Limits tests conducted on the clay till are presented in Figure 1 (Appendix A). The till plots as a clay of low to medium plasticity with liquid limits of 22 to 41 and plastic limits of 11 to 16. The results of particle size distribution analyses are presented in Figure 2.

The deposit was 31.7 and 32.3 m thick, occasionally containing layers of loose to dense silt. The clay till was not penetrated in boreholes 87-1, 87-2, 87-5 and 87-6 which were terminated at depths of 5.0 to 9.6 m (elevation 172.9 to 177.3).

Bedrock

Limestone bedrock was contacted below the clay till at the following depths and elevations confirmed by rock coring:

Location	Depth to Rock (m)	Bedrock Elevation
West Abutment, North Side	33.4	149.7
East Abutment, South Side	32.5	150.3

Rock core description is provided in Table I (Appendix A). The measured core recovery varied between 91 and 100%. The RQD determined from the rock cores was in a typical range of 91 to 100%, indicating an excellent quality rock. A 200 mm thick zone located 1.0 m below the bedrock surface in borehole 87-3 exhibited an RQD of 0%. No loss of drill water circulation was experienced in the process of coring.

The unconfined compressive strength of the rock determined on two representative samples from borehole 87-4 corresponding to depths of 32.6 and 34.3 m (elevation 150.2 and 148.8) was 71 and 66 MPa respectively.

Groundwater

Groundwater was measured in borehole 87-3 at a depth of 0.3 m (elevation 182.8) one day following completion of augering prior to rock coring. Water was not observed in the remaining boreholes during or upon completion of the field work.

Approximately seven weeks after drilling, water was measured at a depth of 0.9 m (elevation 182.3) in the piezometer installed in borehole 87-2.

Based on the observed water levels, 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 water level at this site is expected to be near elevation 182.3.

Groundwater levels may fluctuate subject to seasonal variations and precipitation patterns.

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 Elite Drilling and 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

Table 1 – Rock Core Description

Figure 1 – Plasticity Chart

Figure 2 – Grain Size Distribution Chart

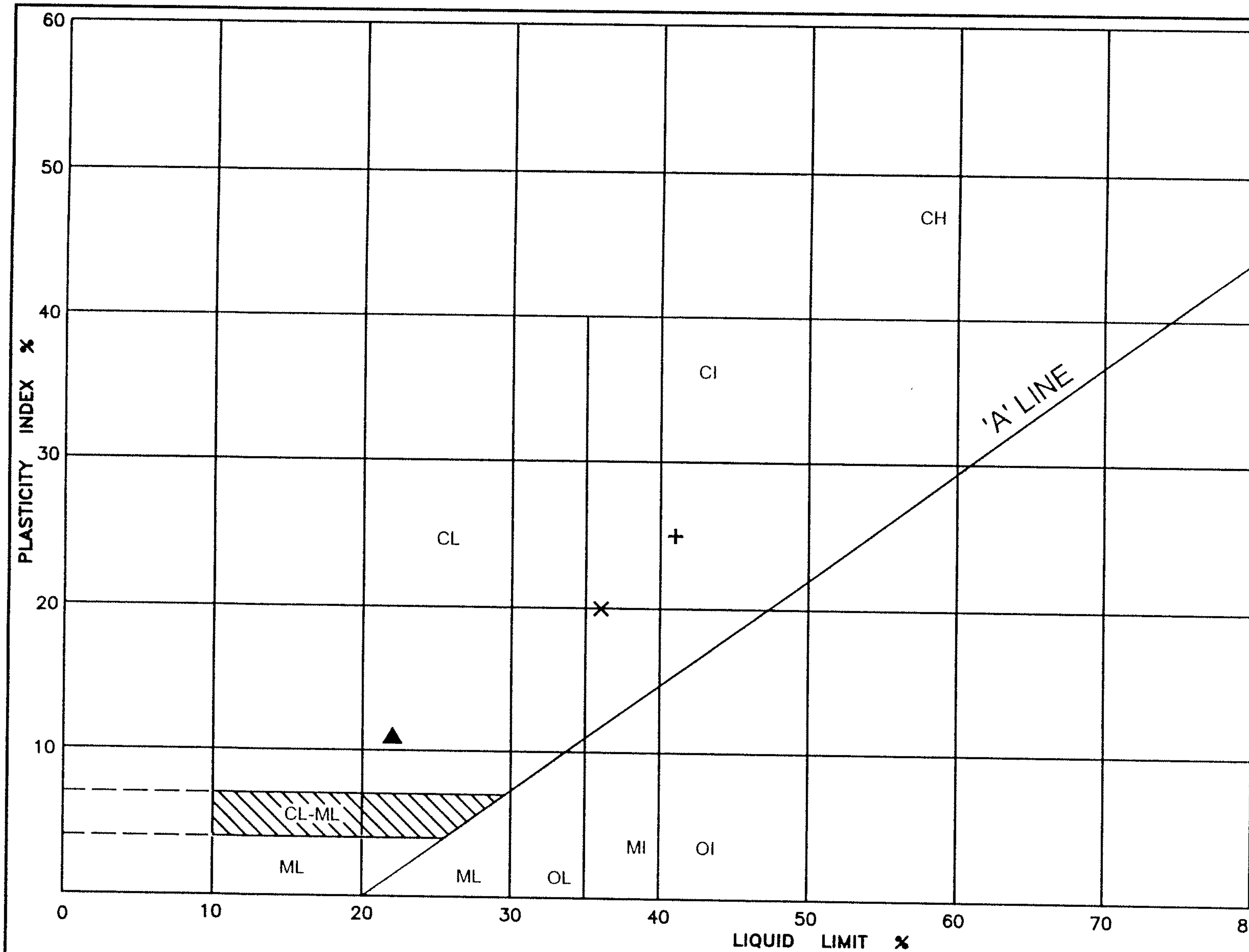
Peto MacCallum Ltd.
CONSULTING ENGINEERS

TABLE I

ROCK CORE DESCRIPTION
ST. JOACHIM ROAD OVERPASS
G.W.P. 60-00-00, SITE 6-87
HIGHWAY 401
TOWN OF LAKESHORE, ONTARIO

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	RUN NO.	DEPTH (m)	RECOVERY %	RQD %	DEPTH (m)	DESCRIPTION
87-3	18	33.40 – 34.90	97	93	33.40 – 36.40	LIMESTONE: grey, aphanitic to fine grained; medium to high strength; unweathered; with occ. white mottling, widely spaced flat bedding layers; fracture index 3; excellent quality
	19	34.90 – 36.40	98	95		
87-4	18	32.50 – 33.50	100	100	32.50 – 35.40	LIMESTONE: grey, aphanitic to fine grained; medium to high strength; unweathered; with occ. white mottling, widely spaced flat bedding layers; fracture index 3; excellent quality
	19	33.50 – 33.70	100	0		
	20	33.70 – 35.40	91	91		

Originated: JFW
Compiled: GD
Checked: MRA



LEGEND

BH	SAMPLE	SYMBOL
87-4	4	×
87-4	11	+
87-4	15	▲

Ministry of Transportation Ontario

PLASTICITY CHART

SILTY CLAY, trace to some sand, trace of gravel (CL-CI)

FIG No 1

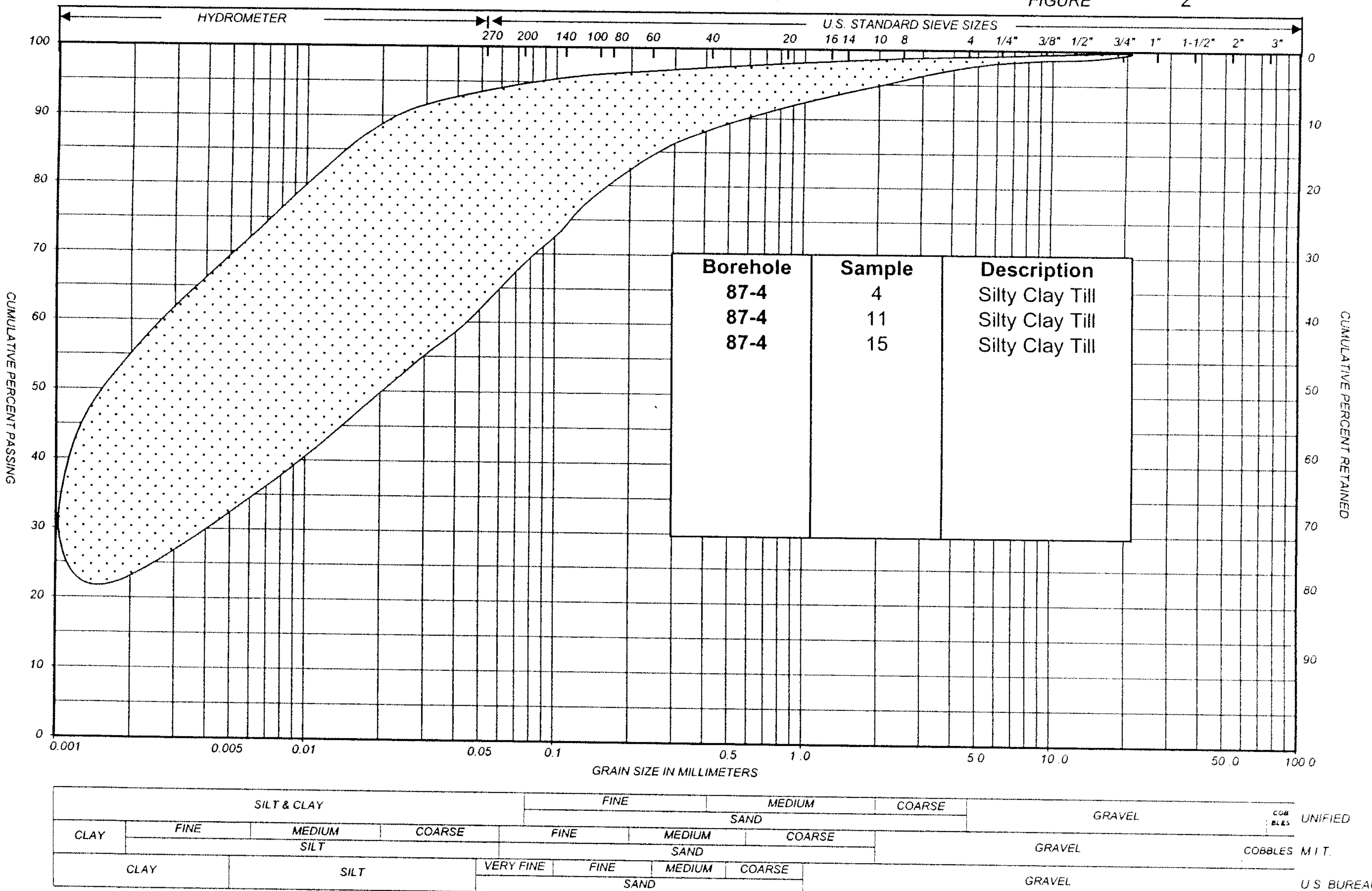
HIGHWAY 401

G.W.P. 60-00-00, Site 6-87

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PML REF.	01TF073G
G.W.P.	60-00-00
FIGURE	2

PARTICLE SIZE DISTRIBUTION CHART



REMARKS SILTY CLAY TILL, trace to some sand, trace of gravel

APPENDIX B

Record of Borehole Sheets

Drawings 1 and 2

Peto MacCallum Ltd.
CONSULTING ENGINEERS

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:

CONSISTENCY	'N' BLOWS/0.3 m	c kPa	DENSENESS	'N' BLOWS/0.3 m
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		

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

■ - UNDRAINED SHEAR STRENGTH DETERMINED FROM POCKET PENETROMETER TEST.

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 87-1

 METRIC

G.W.P. 60-00-00	LOCATION [illegible] N; [illegible]	ORIGINATED BY MB
DIST 31 HWY 101	BOREHOLE TYPE Continuous Flight Solid Stem Augers	COMPILED BY [illegible]
DATUM Geodetic	DATE March [illegible], 2002	CHECKED BY MRA

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	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 (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.39	Ground Level										
0.00	Silty clay Dark Brown Topsoil						182				
0.20	Silty clay, trace to some sand, trace of gravel Stiff Brown (Till)		1	SS	9						
			2	SS	14		181				
	Hard						180				
			3	SS	42		179				
	Very Stiff Grey						178				
177.34			4	SS	16						
5.05	End of Borehole										
	Borehole dry on completion of drilling										

ON_MOT 01TF073G.GPJ ON_MOT.GDT 11/21/2002 2:16:56 PM

$+^7$, $\times^5$ Numbers refer to Sensitivity

20 / 15 —○— 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 87-2

 METRIC

G.W.P. 60-00-00 | LOCATION Co-ords. 4 677 047 N; 293 463 E. | ORIGINATED BY MR

DIST 31 HWY 401 | BOREHOLE TYPE Continuous Flight Solid Stem Augers | COMPILED BY GD

DATUM Geodetic | DATE March 27, 2002 | CHECKED BY MPA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS
183.21	Ground Level					
0.00 / 0.15	Silty clay, Dark Brown (Topsoil)				183	
	Silty clay, trace to some sand, trace of gravel; Stiff; Brown (Till)	1	SS	14	182	
		2	SS	14	181	
	Hard	3	SS	42	180	
	Very stiff	4	SS	21	179	210
	Grey	5	SS	18	178 / 177	
		6	SS	16	176	
	Stiff	7	SS	11	175 / 174	
173.61 / 9.60	End of Borehole					

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; 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 (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Borehole dry on completion of drilling

■ Penetrometer Test

▼ Water level measured after drilling

Piezometer Reading:

Date	Depth (m)
May 14/02	0.95

$+^{7}, \times^{5}$: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 87-3

METRIC

G.W.P. [illegible]	LOCATION [illegible]	ORIGINATED BY [illegible]	
DIST 21	HWY [illegible]	BOREHOLE TYPE [illegible]	COMPILED BY [illegible]
DATUM Geodetic	DATE [illegible]	CHECKED BY MPA	

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
183.09	Ground Level					
0.00	Fine to coarse sand and gravel, some silt Brown (Fill)				▼	183
182.34 0.75	Silty clay, trace of sand Stiff Brown	1	SS	8		182
181.39 1.70	Mottled Dark Brown and Bluish Grey (Fill)	2	SS	9		181
	Silty clay, trace to some sand, trace of gravel Stiff to Hard Brown (Till)	3	SS	16		180
		4	SS	39		
		5	SS	32		179
	Grey	6	SS	15		178
		7	SS	11		177
						176
		8	SS	8		175
			FV			
		9	SS	11		174
	layers of silt, trace of sand and sandy silt, trace of clay and gravel Dense Grey					173
		10	SS	41		172
		11	SS	9		171
						170
	Firm					169
168.09	Cont'd					

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

1.4
172

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

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

$+^{7}, \times^{5}$: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 87-3

 METRIC

G.W.P. 60-30-00 [illegible]	LOCATION [illegible]	ORIGINATED BY MR
DIST [illegible] HWY [illegible]	BOREHOLE TYPE [illegible] & NXL Rock Coring	COMPILED BY [illegible]
DATUM Geodetic	DATE January 1[illegible], 1[illegible] & 1[illegible], 200[illegible]	CHECKED BY MRA

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
18[illegible].[illegible]9	Ground Level										
15.[illegible]0	Silty clay, trace to some sand, trace of gravel Firm Grey (Till		12	SS	4		168		o		
				FV				+			
							167				
							166				
							165				
			13	SS	6				o		
							164				
							163				
							162				
			14	SS	4				o		
							161				
							160				
							159				
			15	SS	3				o		
							158				
							157				
							156				
	occ. thin layers of silt		16	SS	6				o		
							155				
							154				
	Cont'd										

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

$+^7, \times^5$: Numbers refer to Sensitivity — 20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd. Foundation Design

RECORD OF BOREHOLE No 87-3

METRIC

G.W.P. 60-00-00 | LOCATION [illegible] | ORIGINATED BY [illegible]

DIST 31 | HWY [illegible] | BOREHOLE TYPE [illegible] Rock Coring | COMPILED BY [illegible]

DATUM Geodetic | DATE January [illegible], 19 & 19, 200[illegible] | CHECKED BY MRA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
183.09	Ground Level						
	Silty clay, trace to some sand, trace of gravel Firm Grey (Till) (Cont'd)	17	SS	5	152	~17	
					151		
					150		
149.69 / 33.40	Bedrock Unweathered, strong limestone Grey	18	RC	REC 9[illegible]%	149		RQD=93%
		19	RC	REC 98%	148		RQD=95%
146.69 / 36.40	End of Borehole				147		

Water level observed after augering, prior to coring

■ Penetrometer Test

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 (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^7, \times^5$: Numbers refer to Sensitivity; 15–○–5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd. Foundation Design

RECORD OF BOREHOLE No 87-4

 METRIC

G.W.P. [illegible]-00-00 LOCATION [illegible]

DIST [illegible] HWY [illegible] BOREHOLE TYPE [illegible], Rotary and NQ Rock Coring

DATUM Geodetic DATE March 28 & April 1, 2002

ORIGINATED BY [illegible] COMPILED BY [illegible] CHECKED BY MRA

SOIL PROFILE / SAMPLES

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.76	Ground Level				
0.00	Topsoil				
0.23	Silty clay, trace to some sand, trace of gravel; Stiff; Brown (Till)	1	SS	10	
		2	SS	8	
	Hard	3	SS	34	
		4	SS	34	2 16 44 38
	Very Stiff; Grey	5	SS	19	
		6	SS	16	
	Stiff	7	SS	11	
		8	SS	11	
			FV		
		9	SS	10	
	Firm	10	SS	4	
			FV		
		11	SS	7	1 4 40 55
			FV		
	Stiff				
167.76	Cont'd				

GROUND WATER CONDITIONS

ELEVATION SCALE: 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168

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

Field vane annotations: 165, 1.8; 1.6; 1.4

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

+7, ×5: Numbers refer to Sensitivity

15 –○– 5 (with 20 above and 10 below): (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 87-4

 METRIC

G.W.P. [illegible] LOCATION [illegible] ORIGINATED BY MF

DIST [illegible] HWY [illegible] BOREHOLE TYPE [illegible] COMPILED BY [illegible]

DATUM Geodetic DATE March 28 & April 1, [illegible] CHECKED BY [illegible]

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.76	Ground Level						
15.00	Silty clay, trace to some sand, trace of gravel Stiff Grey (Till)	12	SS	[illegible]			
					167		
					166		
	occ. thin layers of silt				165		
		13	SS	12			
					164		
					163		
					162		
		14	SS	12			
					161		
					160		
					159		
		15	SS	13			2 29 46 23
					158		
	Very Stiff				157		
					156		
		16	SS	21			
					155		
					154		
	Stiff						
	Cont'd				153		

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE 20 40 60 80 100 | PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L | WATER CONTENT (%) 20 40 60 | UNIT WEIGHT γ kN/m³

$+^3, \times^5$: Numbers refer to Sensitivity 15—○—5 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd. Foundation Design

RECORD OF BOREHOLE No 87-4

METRIC

G.W.P. [illegible] | LOCATION Co-ords. [illegible] N; [illegible] E. | ORIGINATED BY [illegible]

DIST [illegible] HWY [illegible] | BOREHOLE TYPE C.F.H.S.A., Rotary and NQ Rock Coring | COMPILED BY [illegible]

DATUM Geodetic | DATE March 28 & April 1, 2002 | CHECKED BY [illegible]

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, X LAB VANE; 20, 40, 60, 80, 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20, 40, 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.76	Ground Level										
	Silty clay, trace to some sand, trace of gravel Stiff Grey (Till) (Cont'd)		17	SS	13		152 151				
150.26 32.50	Bedrock Unweathered, strong limestone Grey		18	RC	REC 100%		150				RQD=100%
			19	RC	REC 100%		149				RQD=0%
			20	RC	REC 91%		148				RQD=91%
147.36 35.40	End of Borehole										

Borehole dry on completion of drilling

■ Penetrometer Test

$+^{7}, \times^{5}$: Numbers refer to Sensitivity

20 / 15 – 5 / 10: (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 87-5

METRIC

G.W.P. 00-00-00 | LOCATION [illegible] | ORIGINATED BY MP

DIST 31 | HWY 401 | BOREHOLE TYPE Continuous Flight Solid Stem Augers | COMPILED BY SD

DATUM Geodetic | DATE March 28, 2002 | CHECKED BY MFA

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
182.53	Ground Level				
0.00 / 0.15	Silty clay, Dark Brown (Topsoil)				182
	Silty clay, trace to some sand, trace of gravel; Stiff; Brown (Till)	1	SS	10	
		2	SS	11	181
	Hard	3	SS	46	180
		4	SS	51	179
	Very Stiff; Grey	5	SS	25	
		6	SS	18	178
					177
		7	SS	17	176
	Stiff				175
		8	SS	13	174
172.93 / 9.60	End of Borehole	9	SS	12	173

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 (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

■ Penetrometer Test

Borehole dry on completion of drilling

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

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

RECORD OF BOREHOLE No 87-6

METRIC

G.W.P 60-00-00 LOCATION [illegible] ORIGINATED BY [illegible]

DIST 31 HWY 401 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY [illegible]

DATUM Geodetic DATE March 28, 2002 CHECKED BY [illegible]

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)
182.15	Ground Level					182	
[illegible] 0.15	Silty clay Dark Brown Topsoil						
	Silty clay, trace to some sand, trace of gravel Stiff to Hard Brown (Till)	1	SS	8		181	o (~20)
		2	SS	33		180	o (~18)
	Grey	3	SS	32		179	o (~19)
		4	SS	19		178	o (~21)
177.10 5.05	End of Borehole Borehole dry on completion of drilling						

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 (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

$+^{7}$, $\times^{5}$: Numbers refer to Sensitivity — 20, 15, 5, 10 (%) STRAIN AT FAILURE

METRIC

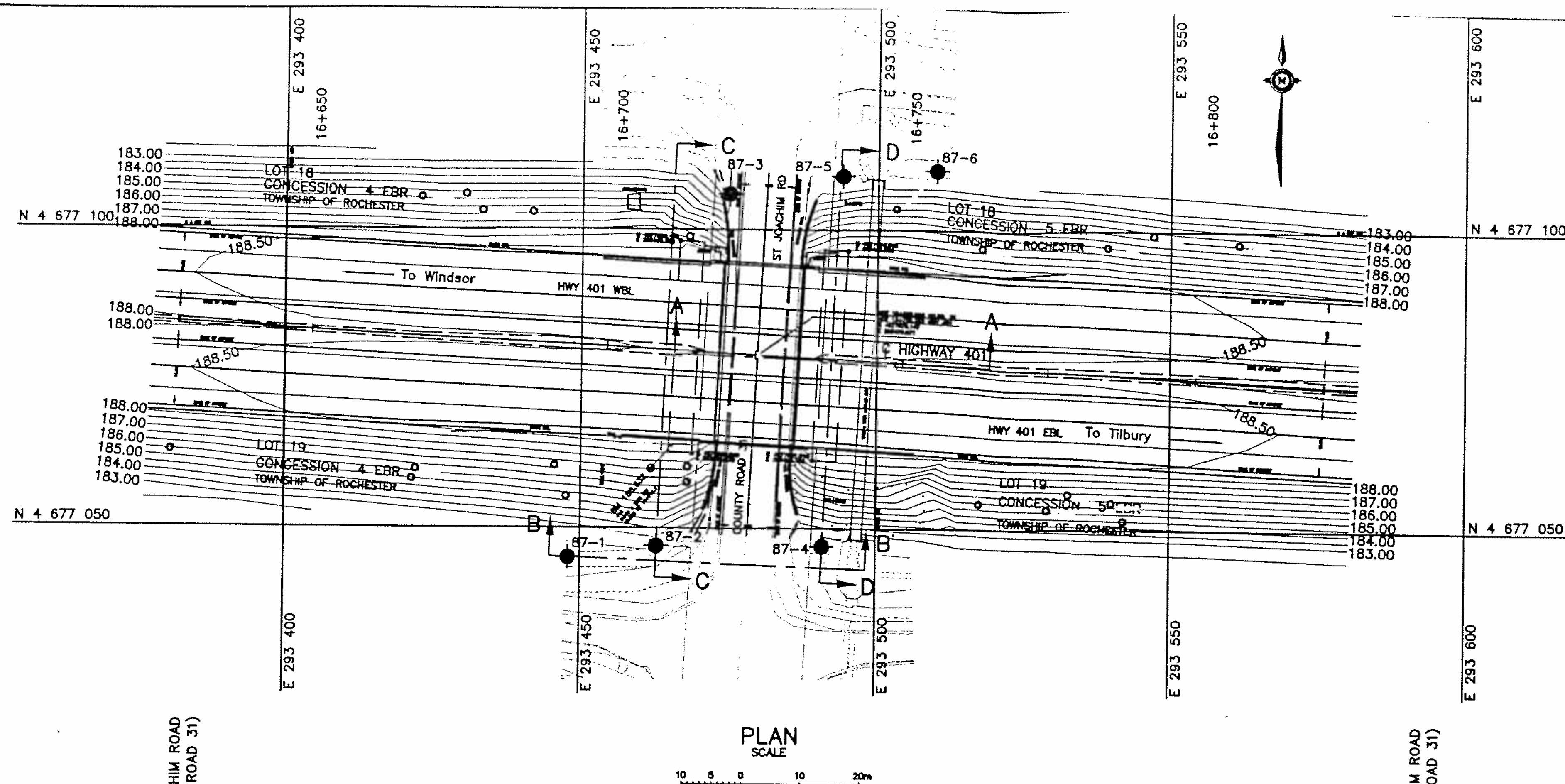
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES

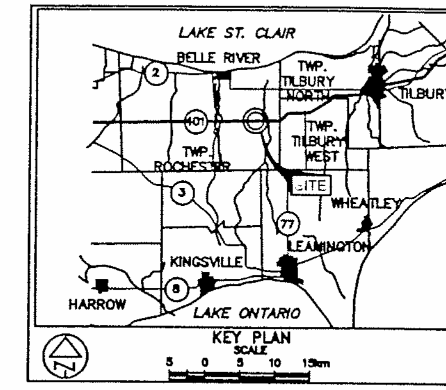
CONT No
GWP No 60-00-00

HIGHWAY 401
PROPOSED OVERPASS AT ST JOACHIM ROAD
BOREHOLE LOCATIONS & SOIL STRATA

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS

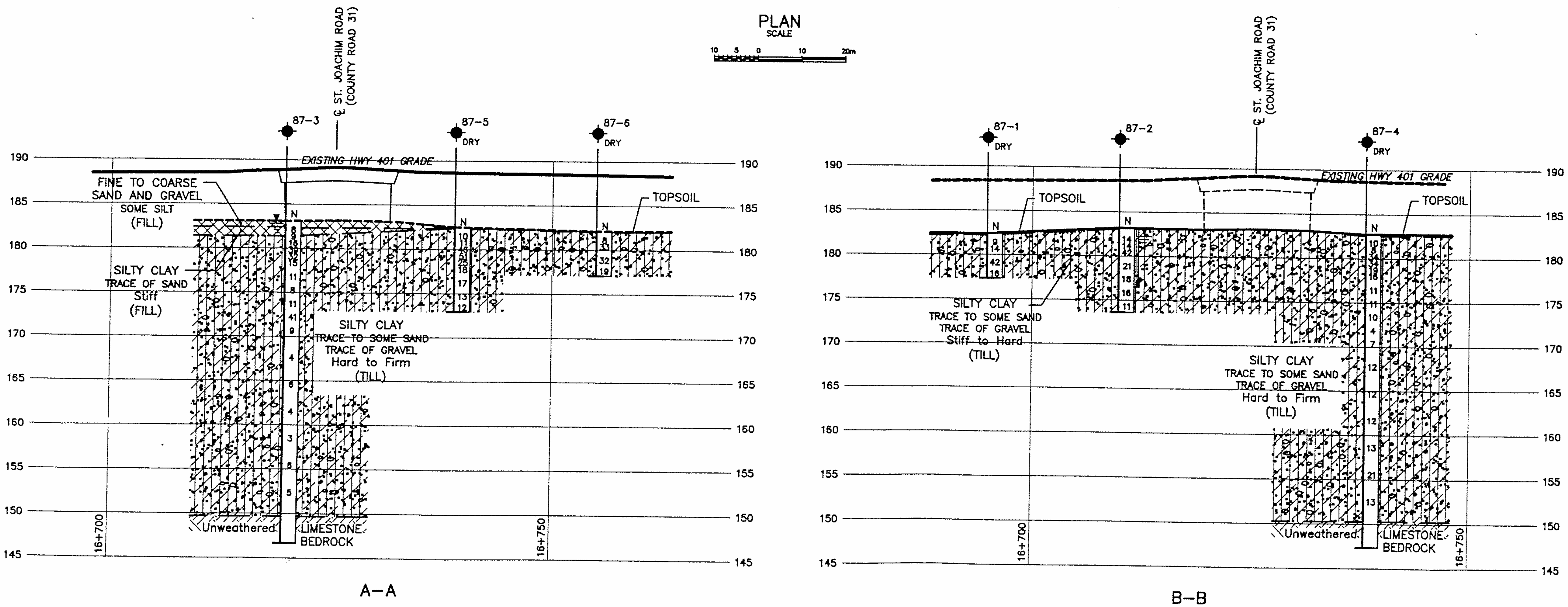




LEGEND

- 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 Jan to Apr 2002
- Head
- ARTESIAN WATER Encountered

BH No	ELEVATION	CO-ORDINATES NORTH	EAST
87-1	182.39	4 677 045	293 448
87-2	183.21	4 677 047	293 463
87-3	183.09	4 677 106	293 475
87-4	182.76	4 677 047	293 491
87-5	182.53	4 677 109	293 494
87-6	182.15	4 677 110	293 510



SECTIONS
SCALE
5 2.5 0 5 10m

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

REVISIONS	DATE	BY	DESCRIPTION

NOTES:

1. 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.
2. REFER TO DRAWING 2 FOR SECTIONS C-C AND D-D.

REF No E-31ROC.dwg; January 2001

Geocres No. 40J2-53

HWY No 401			DIST 31
SUBM'D GO	CHECKED MRA	DATE NOV 22, 2002	SITE 6-87
DRAWN MM	CHECKED BRG	APPROVED DWK	DWG 1

METRIC

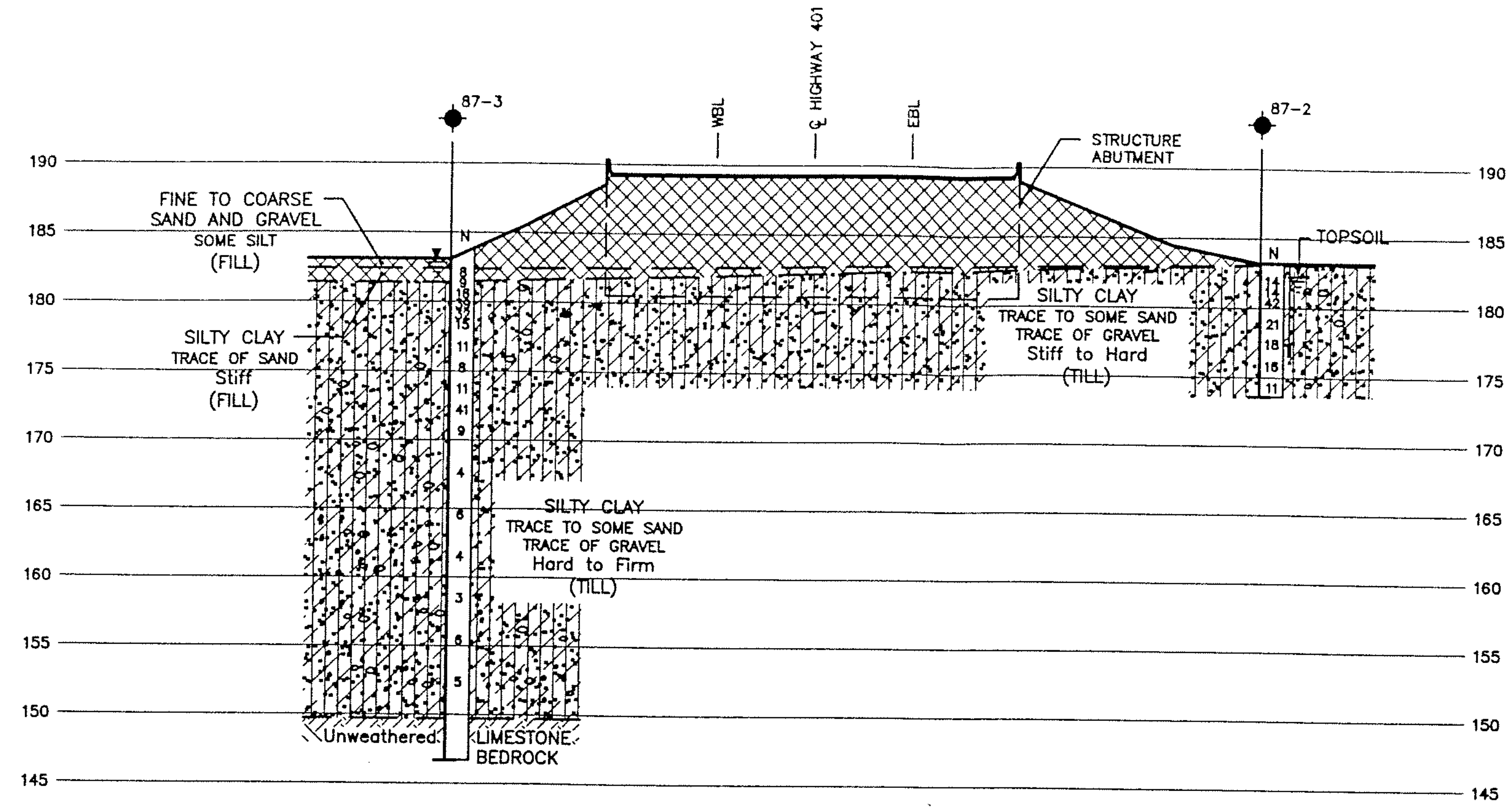
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES

CONT No
GWP No 60-00-00

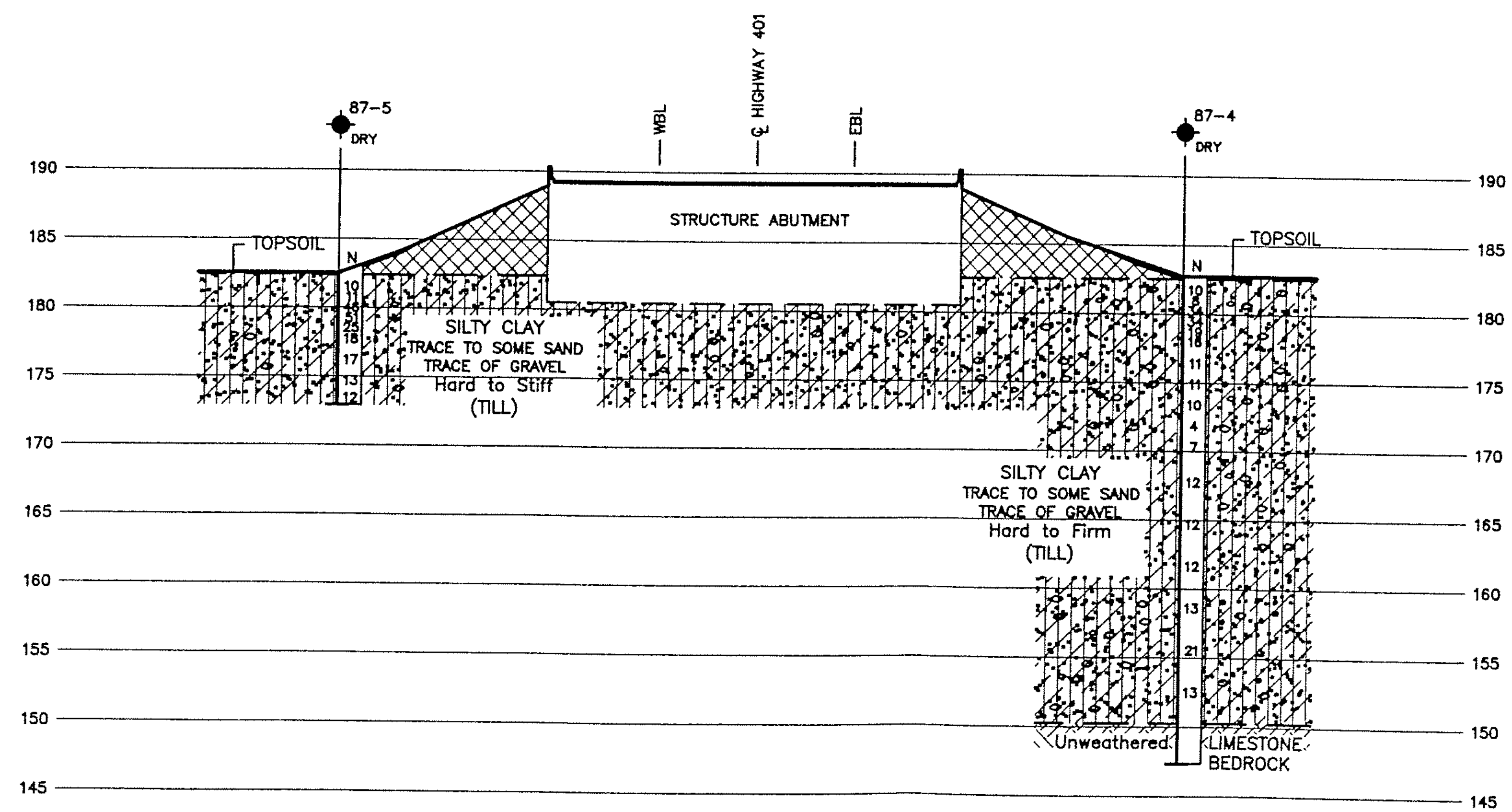
HIGHWAY 401
PROPOSED OVERPASS AT ST JOACHIM ROAD
BOREHOLE LOCATIONS & SOIL STRATA

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS



C–C

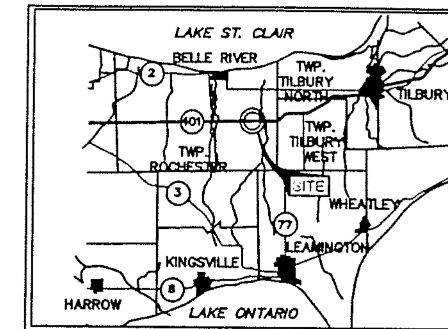


D–D

SECTIONS

SCALE

5 2.5 0 5 10m



KEY PLAN
SCALE
5 0 5 10 15km

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 Jan to Apr 2002
- Head
- ARTESIAN WATER Encountered

BH No	ELEVATION	CO-ORDINATES NORTH	EAST
	(Refer to drawing 1 for co-ordinates)		

– NOTE –

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

REVISIONS DATE	BY	DESCRIPTION

Geocres No. 40J2-53

HWY No 401			DIST 31
SUBM'D GD	CHECKED MRA	DATE NOV 22, 2002	SITE 8-87

NOTES:

1. 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.
2. REFER TO DRAWING 1 FOR PLAN AND SECTIONS A-A AND C-C.

PetoMacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION DESIGN REPORT
FOR
ST. JOACHIM ROAD OVERPASS
G.W.P. 60-00-00, SITE 6-87
HIGHWAY 401
TOWN OF LAKESHORE, ONTARIO

Distribution:

4 cc: Ministry of Transportation
1 cc: Foundation Investigation Report, Ministry of Transportation
1 cc: PML Hamilton
1 cc: PML Toronto

PML Ref: 01TF073G
Geocres No. 40J2-53

November 2002

Peto MacCallum Ltd.
CONSULTING ENGINEERS

TABLE OF CONTENTS

FOUNDATION DESIGN REPORT
for
St. Joachim Road Overpass
G.W.P. 60-00-00, Site 6-87
Highway 401
Town of Lakeshore, Ontario

INTRODUCTION

This report provides geotechnical comments and recommendations regarding design and construction of foundations, abutments and approaches for the proposed overpass structure at St. Joachim Road and Highway 401 in the Town of Lakeshore, Ontario. The investigation was conducted for the Southwestern Region Structural Section of the Ontario Ministry of Transportation.

The proposed overpass will consist of a single span structure of approximately 35 m in width, the span being about 12 m in length (ref. drawing 'Proposed Bridge Site at St. Joachim Road and Highway 401' prepared by the Planning and Design Section of the Ontario Ministry of Transportation's Southwestern Region). Highway 401 will pass over St. Joachim Road at approximate Station 16+725, Highway 401 chainage.

Road grades on St. Joachim Road and Highway 401 at the overpass location will be near elevation 183.2 and 188.9 (see the drawing referred to above) respectively. The approach fills to the structure are about 6 m high.

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised a surficial topsoil underlain by clay till. Limestone bedrock was contacted below the clay till at depths of 32.5 and 33.4 m (elevation 150.3 and 149.7).

FOUNDATIONS

General

The proposed structure can be supported on spread footings or driven piles. Integral abutments supported on driven piles could also be employed. The preferred foundation system will be dictated by structural design considerations, economic considerations and construction constraints.

Piles

Steel H-piles should be driven to refusal on bedrock anticipated at the depths/elevations shown in the following table:

Location	Depth to Rock (m)	Bedrock Elevation
West Abutment	33.4	149.7
East Abutment	32.5	150.3

The recommended factored axial resistance at ultimate limit states (ULS) for two pile sections is presented below:

H-Pile Section	Factored Axial Resistance at ULS (kN)
HP 310 x 79	1450
HP 310 x 110	2000

The resistance at serviceability limit states (SLS) normally allows for 25 mm of compression of the pile and founding medium. Considering the bedrock to be a non-yielding material and the pile length required, the design is not expected to be governed by settlement criteria since the loading required to produce the above deformation of the pile is larger than the factored resistance at ULS.

The soil adjacent to the upper portion of the piles is expected to comprise fill material placed on stiff to very stiff silty clay till. To accommodate movement of the integral abutment system, if employed, it is recommended that two concentric CSPs that extend at least 3 m below the bottom of the abutment be placed around the pile to create an annular space. The inner CSP of 600 mm diameter should be filled with sand meeting the gradation requirements of Granular "B" Type I. Alternatively, a single CSP filled with loose uniform sand meeting the requirements shown in Table I may be used. Refer to MTO Report SO-96-01 for further details.

Since the piles will be about 33 m long, the soil generally comprises stiff to hard clay till, and no evidence of cobbles/boulders was detected during drilling, it is considered, based on our extensive experience with pile driving under similar conditions, that a hammer that **transfers** at least 40 kJ of energy to the pile should be employed to drive the piles. The **rated energy** of the hammer should therefore be 50 to 55 kJ, depending on the type of equipment employed. Since the piles will set on rock, a specific set for this project is not provided.

The piles should be installed and monitored in accordance with the requirements of Special Provision 903S01 (March 2001). This should involve confirmation of the founding elevation, alignment, plumbness, uniformity of set and quality of splices, and should be done on a full-time basis by experienced geotechnical personnel.

Pile caps should be provided with at least 1.2 m of earth cover or equivalent thermal insulation as protection against frost action. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

The following equation should be employed to evaluate the coefficient of horizontal subgrade reaction along the pile:

Granular — Embankment fill above the native clay till surface, elevation 182.5.

$$k_s = n_h z/b$$

where

k_s = coefficient of horizontal subgrade reaction kN/m^3;

n_h = coefficient related to soil density;
= 14,000 kN/m^3;

z = depth, m;

b = pile width, m.

Cohesive — Native soil below the base of the granular fill, elevation 182.5.

$$k_s = \frac{67\, c_u}{b}$$

where

k_s = coefficient of horizontal subgrade reaction kN/m^3;

c_u = undrained shear strength of the clay;
= 65 kPa from ground surface to elevation 181.0;
= 115 kPa elevation 181.0 to 175.0;
= 65 kPa below elevation 175.0;

b = pile width, m.

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile below the annular space. The lateral resistance recommended for the pile sections is as follows:

	HP 310 x 79	HP 310 x 110
Factored Lateral Resistance at ULS	140 kN	190 kN
Lateral Resistance at SLS	35 kN	40 kN

Spread Footings

Supporting the structure on conventional spread footings founded in the native soil is considered to be feasible at this site. Footings founded in the very stiff clay till deposit near 2.0 to 2.5 m depth (elevation 180.5) should be designed using the following resistance values at ultimate and serviceability limit states (assumed footing width of 2.0 m):

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

Spread footings could be constructed on structural fill placed in the approaches. The engineered fill should comprise Granular "A" material placed in maximum 200 mm thick lifts, compacted to 100% standard Proctor maximum dry density, and extended laterally to a line originating at least 1 m from the top of footing and inclined outwards at 45° to the horizontal. This scheme is illustrated in Figure 1. The existing fill and topsoil should be stripped prior to structural fill placement.

The bearing resistance for a minimum 2.5 m wide footing constructed on a minimum 4.0 m thick pad of structural fill is:

Factored Bearing Resistance at ULS	=	900 kPa
Bearing resistance at SLS	=	350 kPa

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.

Caissons

Supporting the structure on augered caissons is not recommended considering the significant depth to bedrock and the potential for squeeze of the clay along the caisson shaft.

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
- γ = unit weight of free-draining granular material (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:

	Granular "A"	Granular "B"
Angle of Internal Friction, degrees	35	32
Unit weight, 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 Passive Earth Pressure K_p	3.69	3.25

Refer to MTO Report SO-96-01 for procedures to determine the earth pressure coefficient to be employed in design of integral abutments. 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:

	Granular "A"	Granular "B"	Silty Clay
Friction Angle, degrees	35	32	0
Cohesion, kPa	0	0	100
Unit weight, kN/m^3	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 of the RSS should be responsible for design of the structure (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 structure should be carried out in conformance with Ontario Provincial Standards specifications for granular backfill (OPSD 3501.00).

The embankments should be constructed in accordance with OPSD 200.010, 202.010 and 208.010. The side slopes of approach fills should be inclined no steeper than 2 horizontal to 1 vertical. Since the embankment fill is less than 8 m high, 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 material are estimated 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 or pile caps is expected to extend through the embankment fill, topsoil and silty clay till. The depth of excavation will vary from about 2.5 m beyond the toe of the embankment fill to as much as 8 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 will be required to support the walls of the excavation and adjacent traffic lanes during construction of the foundations and approach embankment 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 2 and 3 respectively. Recommendations concerning design and construction of the braced excavation support systems are provided in the figures.

A soldier pile and lagging system may 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 stiff to hard clay till. The factored pull-out resistance at ULS of soil anchors in the clay till may be computed as follows:

$$R = 0.45 c_u A_s L_s$$

where c_u = average undrained shear strength over the anchor length

= 100 kPa for 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 2 and 3) provided good quality workmanship and construction practice is employed. The anticipated magnitude of movements is as follows:

	Movement (% of Excavation Depth)
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 4.

The stabilized groundwater level at this site is expected to be near elevation 182.3. Considering the low permeability characteristics of the soil, groundwater seepage or surface water that enters the excavation should be readily handled by conventional sump pumping techniques.

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

Peto MacCallum Ltd.
CONSULTING ENGINEERS

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.

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

REGISTERED PROFESSIONAL ENGINEER
D. W. KERR
PROVINCE OF ONTARIO

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

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

GD:lad

APPENDICES

PML Ref: 01TF073G

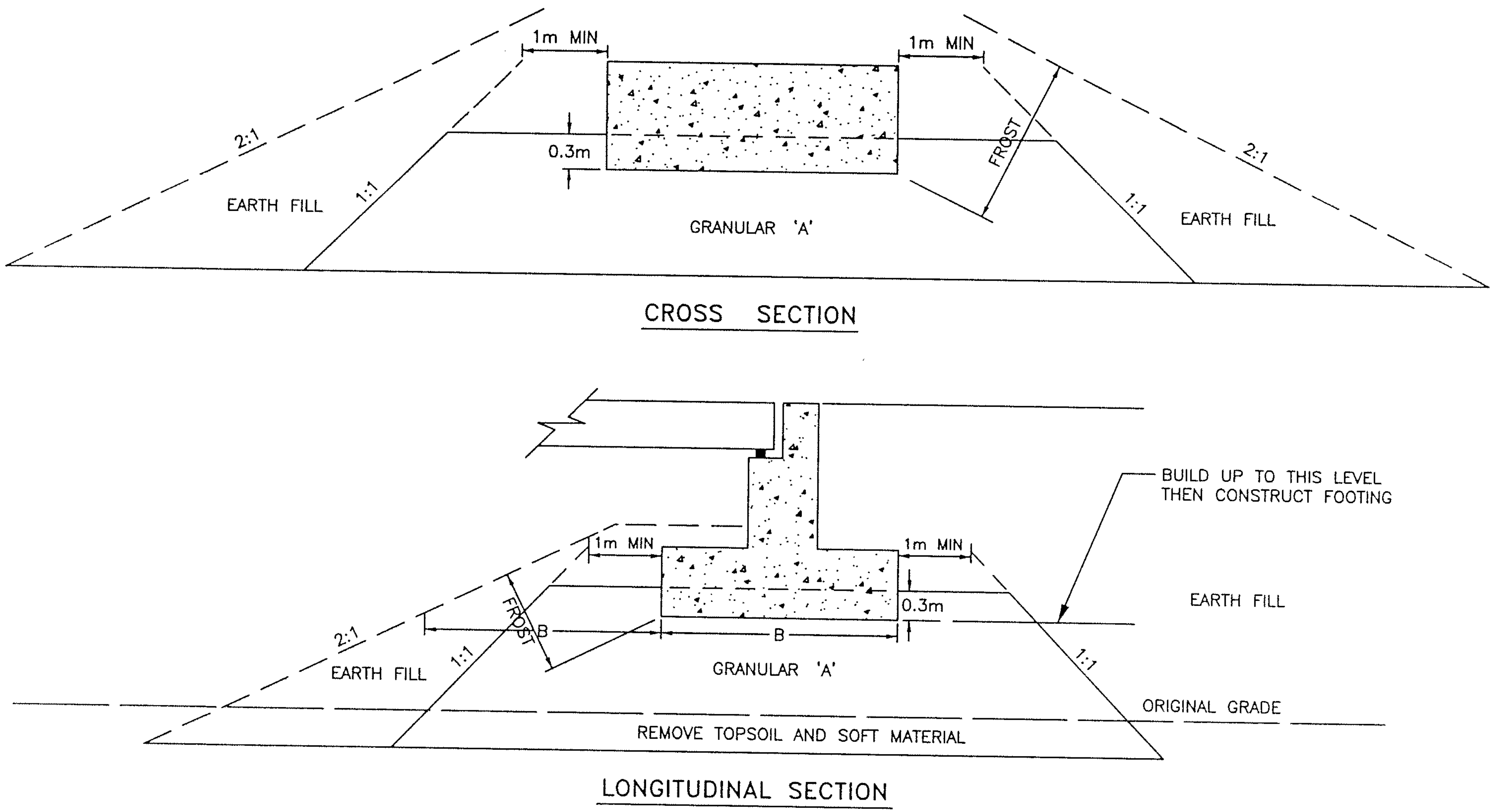
TABLE I

Gradation Specification for Sand Fill in Pre-Augered Holes at Integral Abutments

MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100
600 µm	#30	80 – 100
425 µm	#40	40 – 80
250 µm	#60	5 – 25
150 µm	#100	0 – 6

From MTO Report S0-96-01, Revision 1 – July, 1996.

ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



NOTES

1. REMOVE TOPSOIL AND/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
3. CONSTRUCT CONCRETE FOOTING
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED
5. REFER TO TEXT OF REPORT FOR FROST DEPTH

Peto MacCallum Ltd.
CONSULTING ENGINEERS
45 BURFORD ROAD, HAMILTON, ONTARIO L8E 3C8
Tel: (905) 561-2231 Fax (905) 561-6363

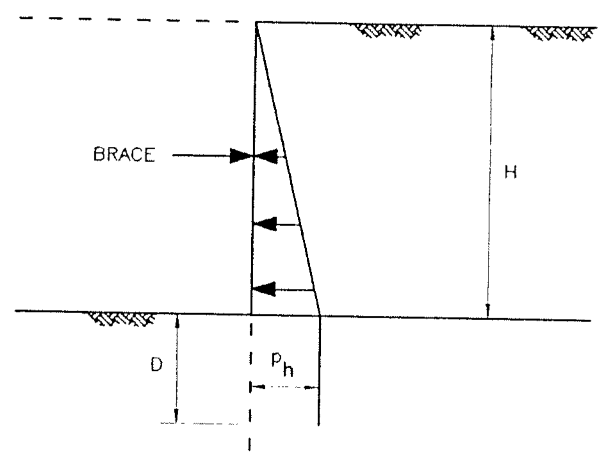
DATE	SCALE	PML REF.	FIGURE NO.
MAY 2002	NTS	01TF073G	1

BC – 007 1/96

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 apppropriate 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 = design lateral earth pressure
= $K\gamma H$

K = lateral earth pressure coefficient

γ = unit weight of soil

H = depth of excavation

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

RECOMMENDED DESIGN PARAMETERS

γ = 20.4 kN/m^3

K = 0.35 (movement of retained soil acceptable)
0.50 (movement of adjacent structures/facilities unacceptable)

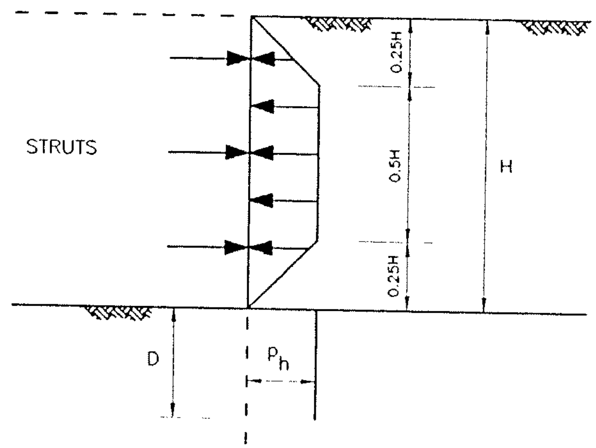
Peto MacCallum Ltd. CONSULTING ENGINEERS	LATERAL EARTH PRESSURE DISTRIBUTION SINGLY BRACED CUTS IN STIFF CLAYS	DATE	JOB NUMBER	FIGURE NO.
		Nov. 2002	01TF073G	2

BC – 004 1/96

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 apppropriate 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 = design lateral earth pressure
= $0.4\,\gamma H$

where

γ = unit weight of soil

H = depth of excavation

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

RECOMMENDED DESIGN PARAMETERS

$\gamma = 20.4\ kN/m^3$

Peto MacCallum Ltd. CONSULTING ENGINEERS	LATERAL EARTH PRESSURE DISTRIBUTION MULTI–BRACED CUTS IN STIFF CLAYS	DATE	JOB NUMBER	FIGURE NO.
		Nov. 2002	01TF073G	3

UP – 001 1/96

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.

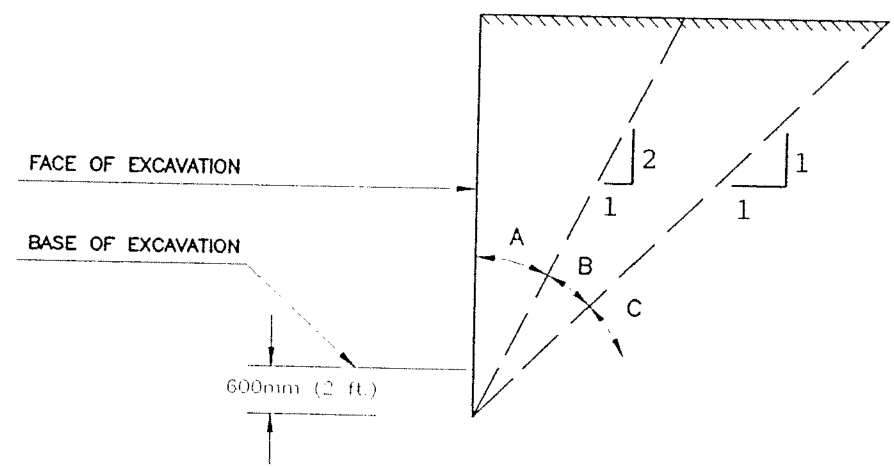
   Underpinning of foundations located in Zones A and B should extend at least into Zone C.

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.



Peto MacCallum Ltd. CONSULTING ENGINEERS

STANDARD DRAWING

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

DATE	JOB NUMBER	FIGURE NO.
Nov. 2002	01TF073G	4