Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
WIDENING OF RUSCOM RIVER BRIDGE
G.W.P. 60-00-00, SITE 6-88
HIGHWAY 401
TOWN OF LAKESHORE, 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: 01TF073H
Geocres No. 40J2-51

November 2002

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for
Widening of Ruscom River Bridge
G.W.P. 60-00-00, Site 6-88
Highway 401
Town of Lakeshore, Ontario

INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed widening of the existing bridge at Ruscom River 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 passes over Ruscom River at approximate Station 18+425, Highway 401 chainage, in the Town of Lakeshore (Township of Rochester). The existing bridge consists of a single span structure with a span of about 25 m and width of 30 m.

The report pertains to the proposed bridge structure 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 Ruscom River. The bridge structure to be widened carries Highway 401 traffic over Ruscom River. At the location of the bridge, Highway 401 runs in the east-west direction.

The site is located in the Township of Rochester in Essex County (Southwestern Ontario), some 35 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 overburden 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 7 to 9, 2002 and comprised four boreholes all advanced to 9.6 m depth at each corner of the bridge at the locations indicated on Drawing 1 (Appendix B).

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 376: Top of round iron bar on south side of Highway 401 west of Ruscom River 44.312 RT 18+326.4
Elevation: 182.147 (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 overburden 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. 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 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. The strata encountered are summarized below.

Fill

Surficial fill was present in all the boreholes. The fill was 1.4 to 2.6 m thick and consisted of stiff silty clay, with a moisture content of 20 to 30%.

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 about 4 m high.

Topsoil

A 400 mm thick layer of topsoil was revealed below the fill at 1.7 m depth (elevation 179.4) in borehole 88-3. Inclusions of topsoil were also identified within the fill in borehole 88-1.

Alluvium

Directly beneath the topsoil in borehole 88-3 was silty clay alluvium containing occasional thin partings of silt and fine sand. This unit was soft to firm and had a thickness of 1.5 m. The moisture content of the unit was 23 and 27%.

Silty Clay Till

Cohesive silty clay till was encountered below the fill (alluvium in borehole 88-3) at depths of 1.4 to 3.6 m (elevation 177.5 to 179.8). The consistency of the clay till was firm to hard, typically stiff to very stiff. Standard penetration test 'N' values ranged from 5 to 32. The results of pocket penetrometer testing carried out in this stratum at various depths indicate values of undrained shear strength in a range of 15 to 185 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 varied between 19 and 25%. The results of the Atterberg Limits tests are presented in Figure 1 (Appendix A). The clay till plots as a clay of medium plasticity, with liquid limits of 37 and 45 and plastic limits of 17 and 20. The results of particle size distribution analyses conducted on the clay till are presented in Figure 2 (Appendix A).

The clay till was not penetrated upon termination of all the boreholes at 9.6 m depth (elevation 171.5 to 172.1).

Clayey Sandy Silt Till

In borehole 88-2, a discontinuous deposit of clayey sandy silt till was revealed within the clay till at a depth of 2.1 m (elevation 179.1). The silt till was compact/very stiff in relative density/consistency, the silt till was 800 mm thick and had a moisture content of about 13%.

Groundwater

Groundwater was measured in boreholes 88-3 and 88-4 at respective depths of 6.5 and 7.5 m (elevation 174.6 and 174.2) at the completion of drilling. Water was not observed in the remaining boreholes.

The water level in Ruscom River in December 2000, indicated on the E-Plan dated May 2001, was approximately elevation 177.5.

Based on these measured water levels along with 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 elevation 178.0.

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 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.

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
PROVINCE OF ONTARIO

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

B. R. GRAY

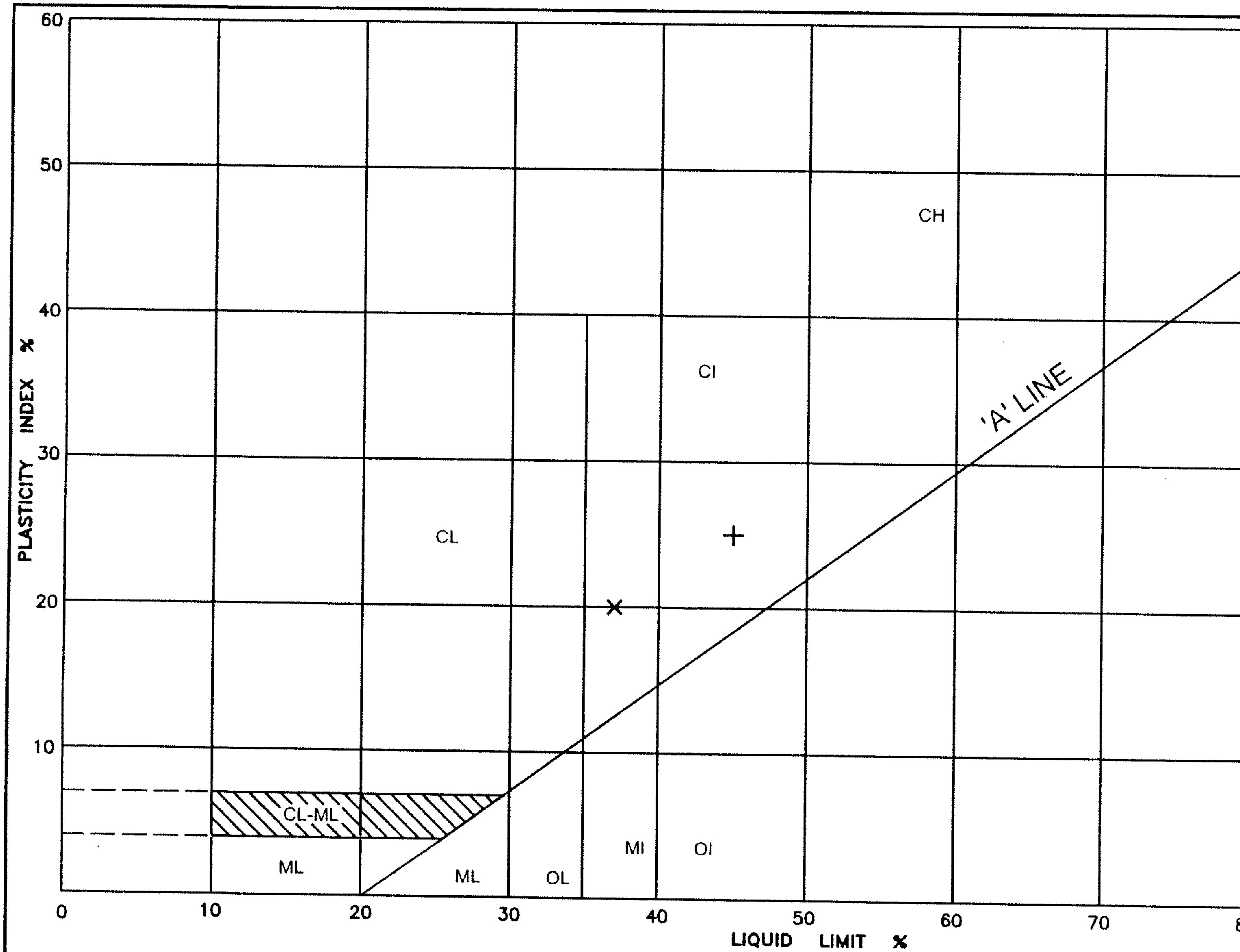
Brian R. Gray, M.Eng., P.Eng.
President

GD:lad

APPENDIX A

Figure 1 – Plasticity Chart

Figure 2 – Grain Size Distribution Chart



LEGEND		
BH	SAMPLE	SYMBOL
88-3	6	+
88-3	9	×

Ministry of Transportation Ontario

PLASTICITY CHART

SILTY CLAY, some sand, trace of gravel (CI)

FIG No 1

HIGHWAY 401

G.W.P. 60-00-00, Site 6-88

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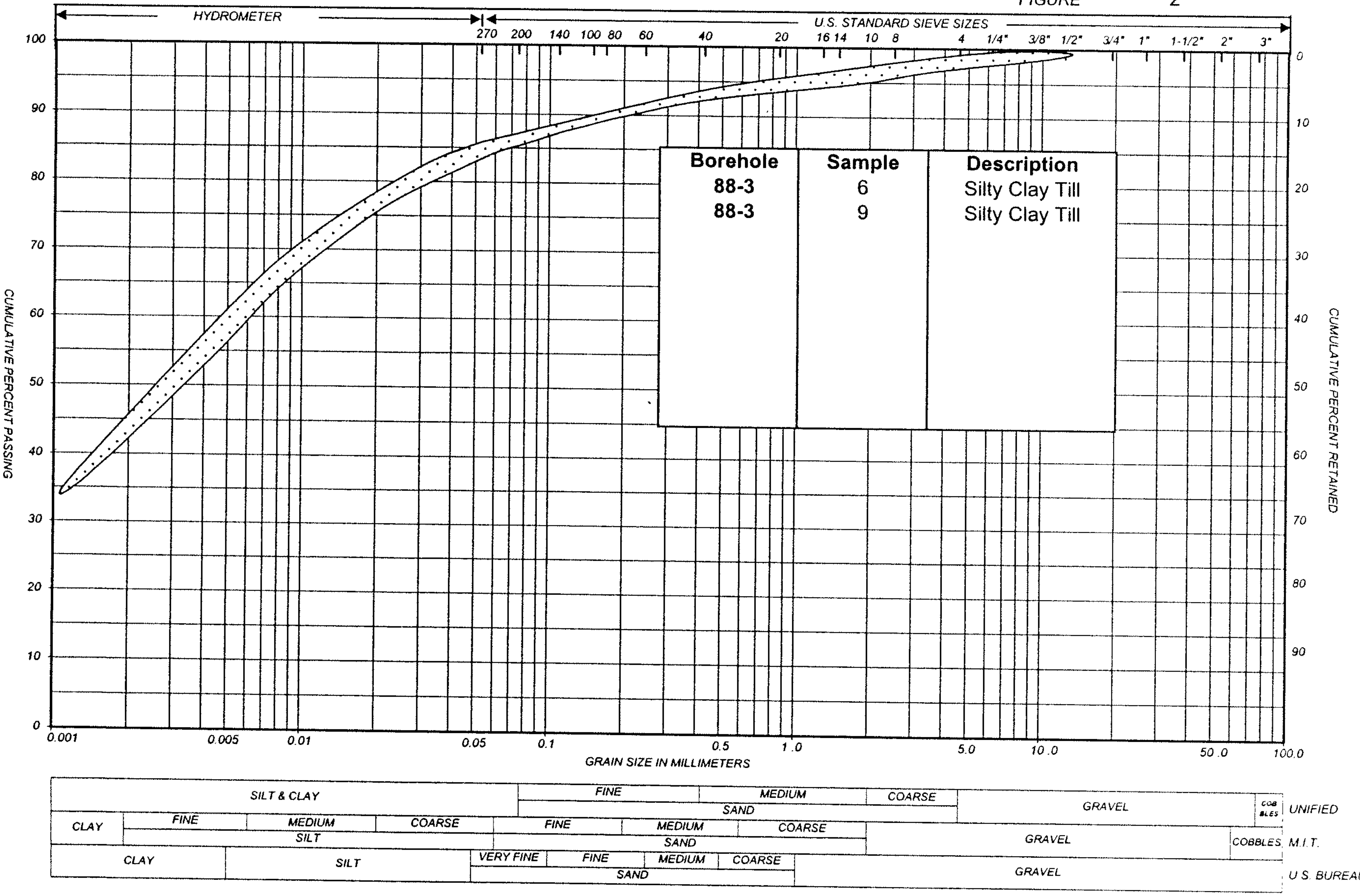
PARTICLE SIZE DISTRIBUTION CHART

PML REF. 01TF073H
G.W.P. 60-00-00
FIGURE 2



Borehole	Sample	Description
88-3	6	Silty Clay Till
88-3	9	Silty Clay Till

REMARKS SILTY CLAY TILL, some sand, trace of gravel (CI)

APPENDIX B

Record of Borehole Sheets

Drawing 1

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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

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RECORD OF BOREHOLE No 88-1

 METRIC

G.W.P.	60-00-00	LOCATION	Co-ords. 4 877 012 N; 295 164 E.	ORIGINATED BY	MR
DIST	31 HWY 401	BOREHOLE TYPE	Continuous Flight Solid Stem Augers	COMPILED BY	JP
DATUM	Geodetic	DATE	May 09, 2002	CHECKED BY	MRA

SOIL PROFILE / SAMPLES

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (Penetrometer Test)
181.19	Ground Level					
0.00	Silty clay, trace of sand; Stiff; Brown (Fill)				181	
		1	SS	10	180	
	with mottling topsoil	2	SS	11	179	
178.59 / 2.60	Silty clay, some sand, trace of gravel; Very Stiff to Hard; Brown (Till)	3	SS	14		
		4	SS	32	178	
		5	SS	25	177	
		6	SS	19	176	125
	Stiff; Grey	7	SS	12	175	
		8	SS	13	174 / 173	
		9	SS	10	172	
171.59 / 9.60	End of Borehole					

End of Borehole

Borehole dry on completion of drilling

■ Penetrometer Test

Column headings: ELEV DEPTH; DESCRIPTION; STRAT PLOT; NUMBER; TYPE; "N" VALUES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_p, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (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

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RECORD OF BOREHOLE No 88-2

METRIC

G.W.P. 60-00-00 LOCATION Coords. 1 676 [illegible]1 N; 295 169 E. ORIGINATED BY MB

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

DATUM Geodetic DATE May 0[illegible], 2002 CHECKED BY MRA

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION
181.24	Ground Level
0.00	Silty clay, trace of sand Stiff Brown Mottled (Fill)
179.84 / 1.40	Silty clay, some sand, trace of gravel Stiff to Very Stiff Brown (Till)
179.14 / 2.10	Clayey sandy silt, trace of gravel Compact/Very Stiff Brown (Till)
178.34 / 2.90	Silty clay, some sand, trace of gravel Very Stiff Brown (Till) Grey Stiff Firm
171.64 / 9.60	End of Borehole Borehole dry on completion of drilling ■ Penetrometer Test

SAMPLE NUMBER	TYPE	"N" VALUES
1	SS	10
2	SS	15
3	SS	30
4	SS	23
5	SS	25
6	SS	20
7	SS	12
8	SS	9
9	SS	7

STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE: 181, 180, 179, 178, 177, 176, 175, 174, 173, 172

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

150

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

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RECORD OF BOREHOLE No 88-3

METRIC

G.W.P. 60-00-00 LOCATION Co-ords. 4 677 007 N; 295 200 E. ORIGINATED BY MR

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

DATUM Geodetic DATE May 09, 2002 CHECKED BY MRA

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	SHEAR STRENGTH kPa	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.12	Ground Level						
0.00	Silty clay, trace of sand Stiff Brown Mottled (Fill)	1	SS	9			
179.42 / 1.70	Silty clay Dark Brown Topsoil	2	SS	9			
179.02 / 2.10	Silty clay, trace of sand Soft to Firm Blueish Grey Alluvium with occ. thin partings of silt and fine sand at 3.0m	3	SS	4			
		4	SS	7			
177.52 / 3.60	Silty clay, some sand, trace of gravel Very Stiff Brown (Till)	5	SS	18		150	
		6	SS	17		125	2 12 43 43
	Stiff Grey	7	SS	13	▼		
		8	SS	13			
		9	SS	12			1 12 41 46
171.52 / 9.60	End of Borehole						

Elevation scale: 181, 180, 179, 178, 177, 176, 175, 174, 173, 172

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

▼ Water level measured after drilling

■ Penetrometer Test

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

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

Ministry of Transportation Ontario

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RECORD OF BOREHOLE No 88-4

METRIC

G.W.P. 60-00-00 LOCATION Co-ords. 4 676 967 N; 295 197 E. ORIGINATED BY MR

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

DATUM Geodetic DATE May 08, 2002 CHECKED BY MRA

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (Penetrometer)	WATER CONTENT (%)	REMARKS
181.72	Ground Level								
0.00	Silty clay, trace of sand; Stiff; Brown Mottled (Fill)	1	SS	13		181			
		2	SS	12		180			
179.82 / 1.90	Silty clay, some sand, trace of gravel; Stiff to Very Stiff; Brown (Till)	3	SS	12		179			
		4	SS	23		178			
		5	SS	20				187	
	Stiff — Grey	6	SS	11		177	■		
		7	SS	9		176 / 175	■		
	Firm — Brown	8	SS	5	▼	174	■		
172.12 / 9.60		9	SS	7		173	■		
	End of Borehole								

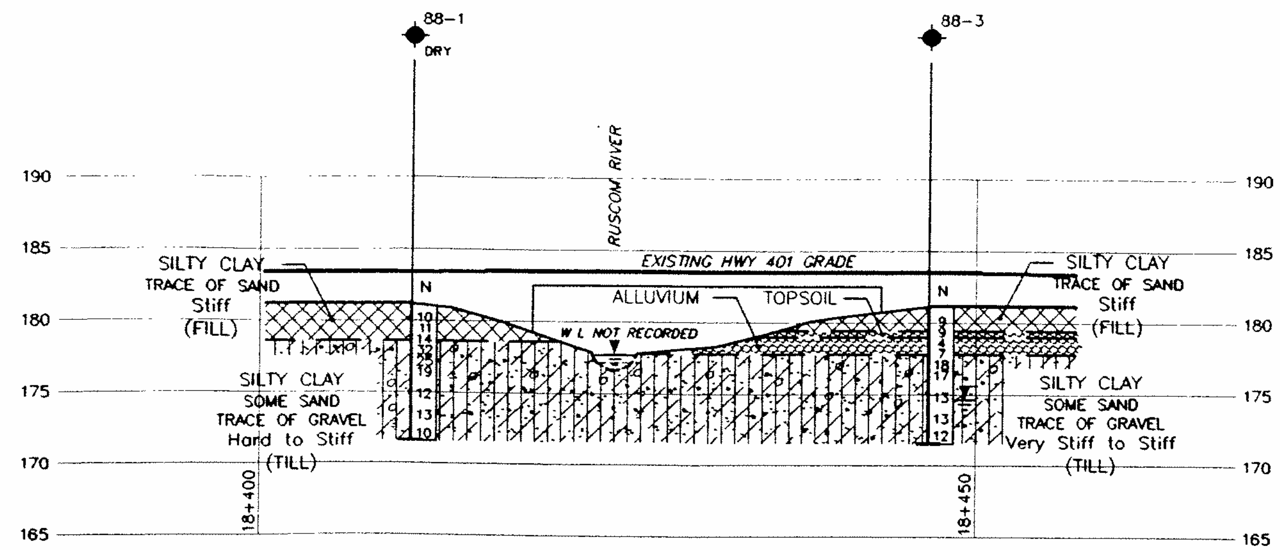
Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC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

▼ Water level measured after drilling

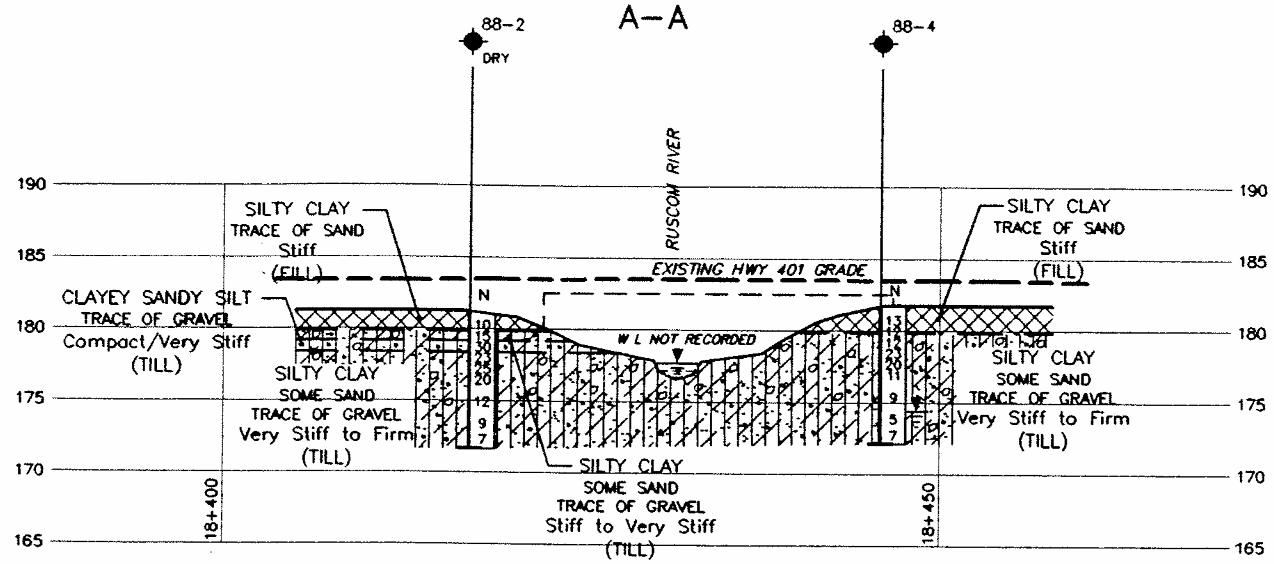
■ Penetrometer Test

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

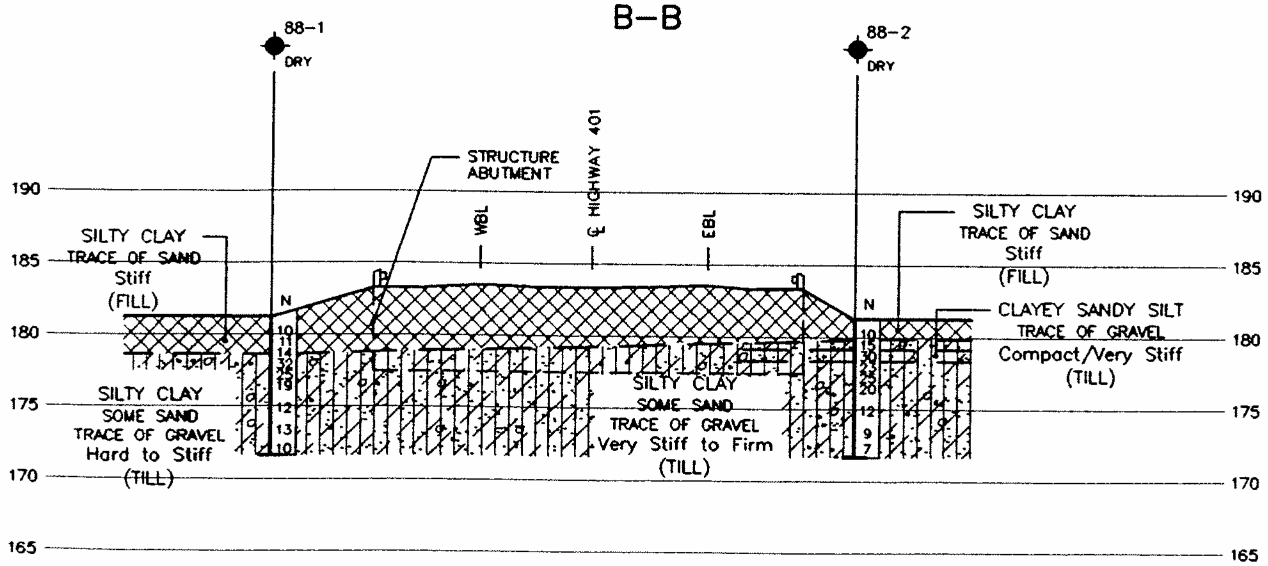
20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE



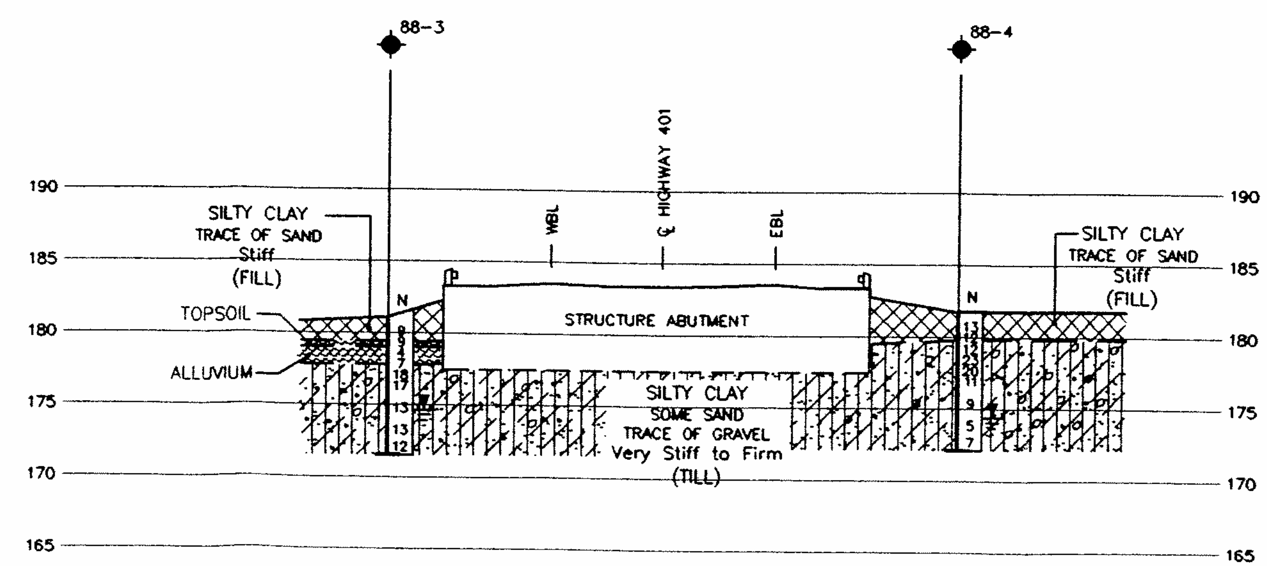
A–A



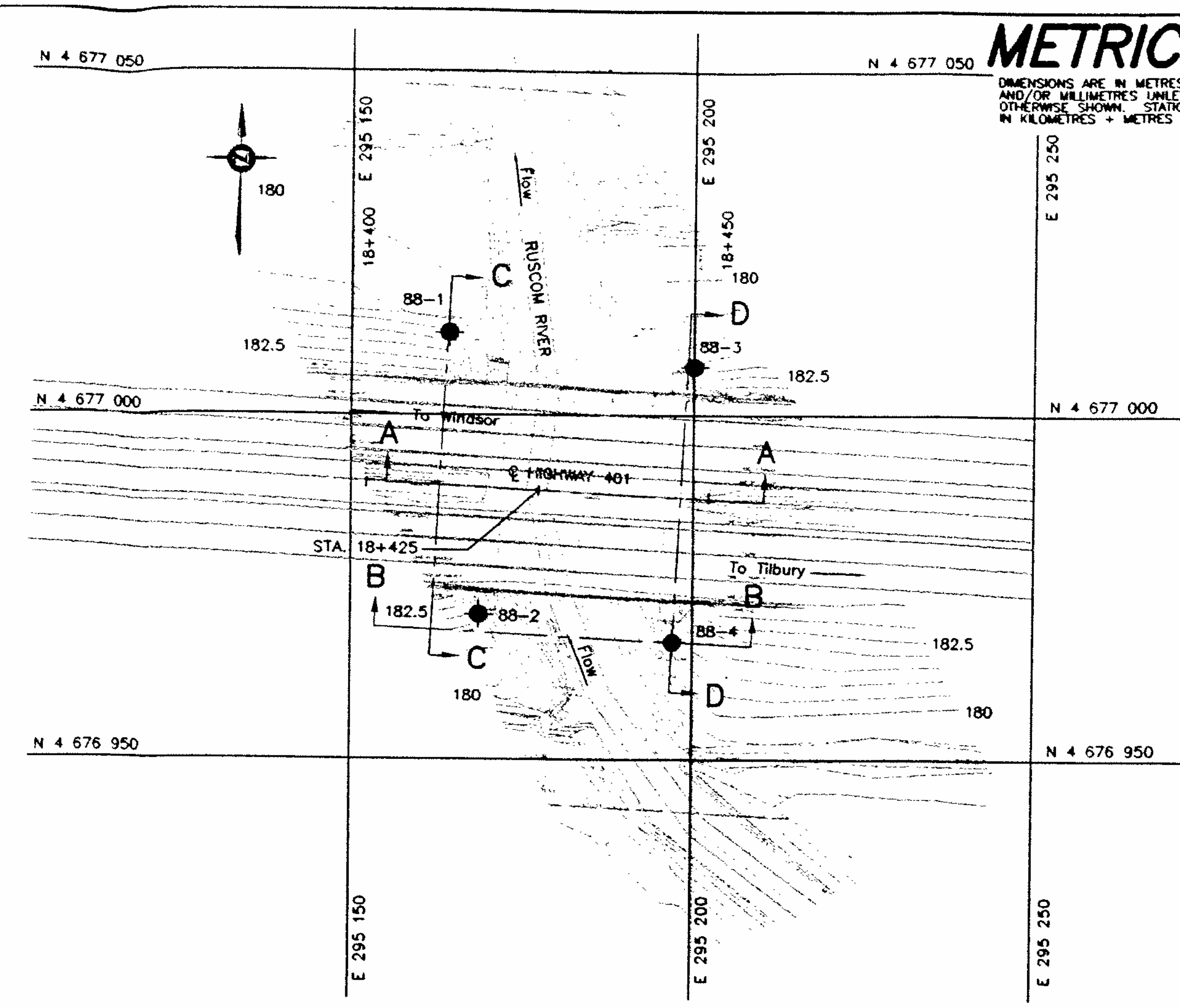
B–B



C–C



D–D



PLAN

SCALE

METRIC

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

CONT No

GWP No 60-00-00

SHEET

HIGHWAY 401

PROPOSED BRIDGE WIDENING AT RUSCOM RIVER

BOREHOLE LOCATIONS & SOIL STRATA

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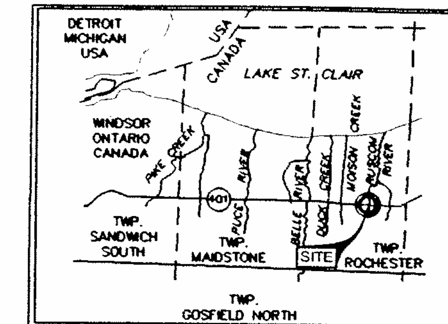

KEY PLAN

SCALE

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
88-1	181.19	4 677 012	295 164
88-2	181.24	4 676 971	295 169
88-3	181.12	4 677 007	295 200
88-4	181.72	4 676 967	295 197

– NOTE –

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

SECTIONS

SCALE

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.

REF No Survey Plan 2001 Site 6–88, entitled Proposed Crossing at Ruscom River and King's Highway 401. Prepared by J.D. Barnes Limited and Planning and Design Section, MTO.

REVISIONS		
DATE	BY	DESCRIPTION

Geocres No.

HWY No	401			DIST	31
SUBM'D	GO	CHECKED	MRA	DATE NOV 22, 2002	SITE 6–88
DRAWN	MM	CHECKED	BRC	APPROVED DWK	DWG 1

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FOUNDATION DESIGN REPORT

for
Widening of Ruscom River Bridge
G.W.P. 60-00-00, Site 6-88
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 widening of the existing bridge at Ruscom River 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 existing bridge consists of a single span superstructure with a width of approximately 30 m and span of about 25 m (ref. drawing No. RSROC-E 'Crossing at Ruscom River and Highway 401 at 18+424.570' dated May 2001). Highway 401 will pass over Ruscom River at approximate Station 18+425, Highway 401 chainage.

Road grade on Highway 401 at the bridge location is near elevation 183.7 (interpolated from existing grade shown on the drawing referred to above). The existing approach embankments to the bridge comprise fill with a height of approximately 4 m. It is understood that the existing bridge is supported on shallow spread footings. Details concerning the design of the foundations supporting the existing structure were not provided.

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised a surficial fill underlain by silty clay till. The clay till exhibits a typically stiff to very stiff crust and becomes stiff to firm below a depth of about 5 m.

FOUNDATIONS

Spread Footings

Supporting the structure widenings on conventional spread footings founded in the native overburden is considered to be feasible at this site. The new footings should be founded at the same level as the existing footings. However, since the founding level of the existing footings is not confirmed, and the clay till exhibits a stiff to very stiff crust over lower strength material, the design bearing resistance will depend upon the width and founding level of the new footings.

Based on the river bed elevation shown on the drawing referred to above 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 177.5.

For a 2.5 m wide strip footing constructed on the stiff to very stiff 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	=	225 kPa
Bearing Resistance at SLS	=	150 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 4 m thick and a minimum 4 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 structure.

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	0.42	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 overburden. 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	75
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 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 overburden 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, topsoil and alluvium layers. Based on the inferred design founding elevation of 177.5, the depth of excavation will vary from about 4.0 m beyond the toe of the embankment fill to as much as 6.0 m at the top of the fill. Cognizant of the fill present at the site, the overburden 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. This may be a particular concern when excavated through the alluvium identified in borehole 88-3.

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.

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 very stiff clay till. The factored pull-out resistance at ULS of soil anchors in the clay till may be computed as follows:

$$R = 0.5 c_u A_s L_s$$

where c_u = average undrained shear strength over the anchor length
= 75 kPa for stiff to very stiff 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:

	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 3.

The stabilized groundwater level at this site is expected to be near elevation 178.0. Considering the low permeability characteristics of the overburden, 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.

Considering the low permeability characteristics of the overburden, groundwater seepage or surface water that enters the excavation should be readily handled by conventional sump pumping techniques. "Flood conditions" may occur during construction and surface water may flow into the excavations. 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.

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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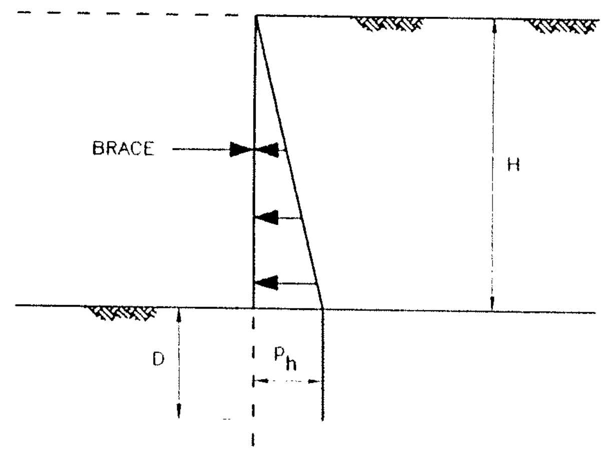
APPENDICES

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

$\gamma = 20.4$ kN/m^3

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

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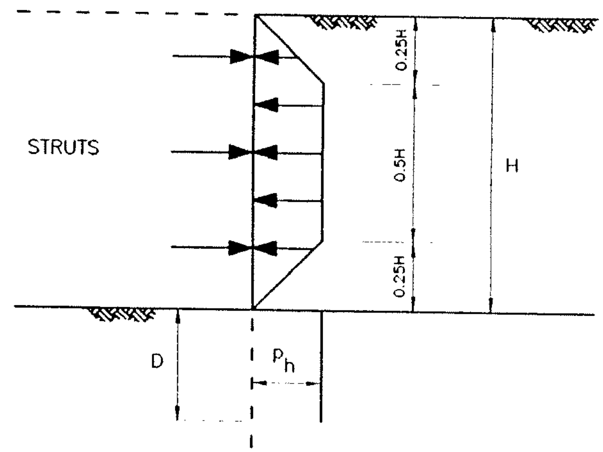
LATERAL EARTH PRESSURE DISTRIBUTION SINGLY BRACED CUTS IN STIFF CLAYS	DATE	JOB NUMBER	FIGURE NO.
	Nov. 2002	01TF073H	1

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	01TF073H	2

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.

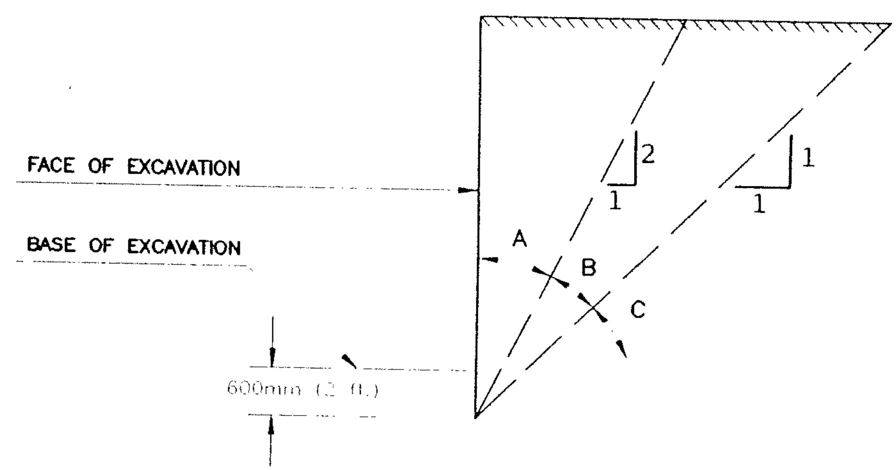
   Underpinning of foundations located in Zones A and B should extend at least into Zone C.

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.



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STANDARD DRAWING	DATE	JOB NUMBER	FIGURE NO.
GENERAL RECOMMENDATIONS REGARDING UNDERPINNING OF FOUNDATIONS/UTILITIES LOCATED CLOSE TO EXCAVATION	Nov. 2002	01TF073H	3