Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION AND DESIGN REPORT FOR WIDENING OF PUCE RIVER BRIDGE G.W.P. 60-00-00, SITE 6-83 HIGHWAY 401 TOWN OF LAKESHORE, ONTARIO

Distribution:

4 cc: Ministry of Transportation
1 cc: Final Investigation Report, Ministry of Transportation
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PML Ref: 01TF073C
Geocres No. 40J2-54

November 2002

Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION REPORT
FOR
WIDENING OF PUCE RIVER BRIDGE
G.W.P. 60-00-00, SITE 6-83
HIGHWAY 401
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FOUNDATION INVESTIGATION REPORT
for
Widening of Puce River Bridge
G.W.P. 60-00-00, Site 6-83
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 Puce 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 Puce River at approximate Station 16+578, Highway 401 chainage, in the Town of Lakeshore (Township of Maidstone). The existing bridge consists of a single span superstructure with a width of approximately 30 m and span of about 10 m.

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

SITE DESCRIPTION

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

The site is located in the Township of Maidstone in Essex County (Southwestern Ontario), some 20 km east of Windsor along Highway 401. The surrounding lands are mainly level and used for a mix of residential and agricultural purposes.

The area is part of the Essex Clay Plain physiographic sub-region. It is essentially a till plain smoothed by deposits of lacustrine clay which settled in the depressions while the knolls were being lowered by wave action. In general, the 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 for the current investigation was carried out on May 13, 2002 and comprised two boreholes drilled to depths of 9.6 and 11.1 m on the north side of the bridge at the locations shown on Drawing 1 (Appendix B).

The Record of Boreholes for two boreholes drilled on the south side of the bridge to depths of 6.6 and 13.1 m (W.P. 693-64) were retrieved from the Geocres files to supplement the subsurface information retrieved during this study.

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 VCP549: Top of round iron bar on south side of Highway 401 east of Puce River 18.868 RT 16+616.445 Elevation 185.322 (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).

The borehole locations were selected to supplement the existing subsurface information obtained during a previous investigation (WP 693-64) conducted on the south side of the bridge in 1965. The existing borehole locations are shown on Drawing 1 (Appendix B). A summary of the boreholes drilled at this site is provided in the following table:

Location	Borehole No.	Depth, m
West Abutment, North Side	83-1 (GWP 60-00-00)	11.1
East Abutment, North Side	83-2 (GWP 60-00-00)	9.6
East Abutment, South Side	1 (WP 693-64)	13.1
West Abutment, South Side	2 (WP 693-64)	6.6

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 and dynamic cone penetration test results, pocket penetrometer and in situ vane shear strength values, 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 logs from the 1965 study were presented in imperial units. The equivalent metric depths and elevations have been added to the Record of Borehole sheets for reference.

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 comprised a surficial fill, silty clay/clayey silt till and discontinuous deposits of silty clay, silt, sandy silt and silty sand. The strata encountered are summarized below.

Fill

Fill was present surficially in borehole 83-1. The fill consisted of two layers – 1.4 m of very stiff silty clay over 0.8 m of very stiff sandy clayey silt.

It is noteworthy that the boreholes were drilled at the toe of slope of the highway embankment fill. The embankment fill at this site is about 2.5 m high.

Silty Clay/Clayey Silt Till

Cohesive silty clay/clayey silt till was encountered below the fill at 2.2 m depth (elevation 182.4) in borehole 83-1 and surficially in the remaining boreholes. The till was stiff to hard and had a thickness of 5.2 to 8.0 m where penetrated. In borehole 1, the till was not penetrated upon completion of augering at a depth of 13.1 m (elevation 170.6).

Standard penetration test 'N' values varied broadly between 9 and 89, typically decreasing below elevation 180.0. The results of pocket penetrometer testing carried out in this stratum at various depths indicate that the undrained shear strength of the till is in a range of 15 to 115 kPa, decreasing with depth. (Values less than about 30 kPa were likely obtained from testing of disturbed samples.) In situ vane shear strength values ranged from 35 to 110 kPa (sensitivity of about 2.0) and shear strength values determined by unconfined compression testing previously reported ranged from 30 to 75 kPa.

The moisture content of the silty clay till ranged from 16 to 24%, locally to 29%. 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 plastic limits of 17 to 19 and liquid limits of 36 to 42. Previous testing indicated plastic and liquid limits of 16 to 22 and 28 to 41, respectively. The results of particle size distribution analyses conducted on the silty clay till are presented in Figure 2 (Appendix A).

Silty Clay

Underlying the silty clay till at 10.2 m depth (elevation 174.4) in borehole 83-1 was cohesive silty clay, its consistency being very stiff (SPT-'N' value of 25). This unit was not penetrated upon termination of the borehole at a depth of 11.1 m (elevation 173.5).

Silt

In borehole 83-2, a deposit of cohesionless silt was revealed below the silty clay till at 7.1 m depth (elevation 175.6). Being compact in relative density (SPT-'N' values of 11 and 30), the silt had a moisture content of about 22% and was not penetrated upon termination of the borehole at a depth of 9.6 m (elevation 173.1).

Sandy Silt/Silty Sand

A 1.0 m thick layer of sandy silt was encountered within the clayey silt till at 6.2 m depth (elevation 177.6) in borehole 1.

Silty sand was revealed as a 150 mm layer within the clayey silt till at a depth of 12.5 m (elevation 171.2) in borehole 1 and below the clayey silt to silty clay till at 5.2 m depth (elevation 178.0) in borehole 2. The latter borehole was terminated within the silty sand at a depth of 6.6 m (elevation 176.6).

Groundwater

Groundwater was measured in borehole 83-2 at a depth of 7.3 m (elevation 175.4) at the completion of drilling. Water was not observed in the remaining boreholes.

The water level measured in the Puce River in January 2001 was at elevation 182.1.

Based on the measured water level 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 the water level in the adjacent river, 182.1 at the time of drilling.

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.

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

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

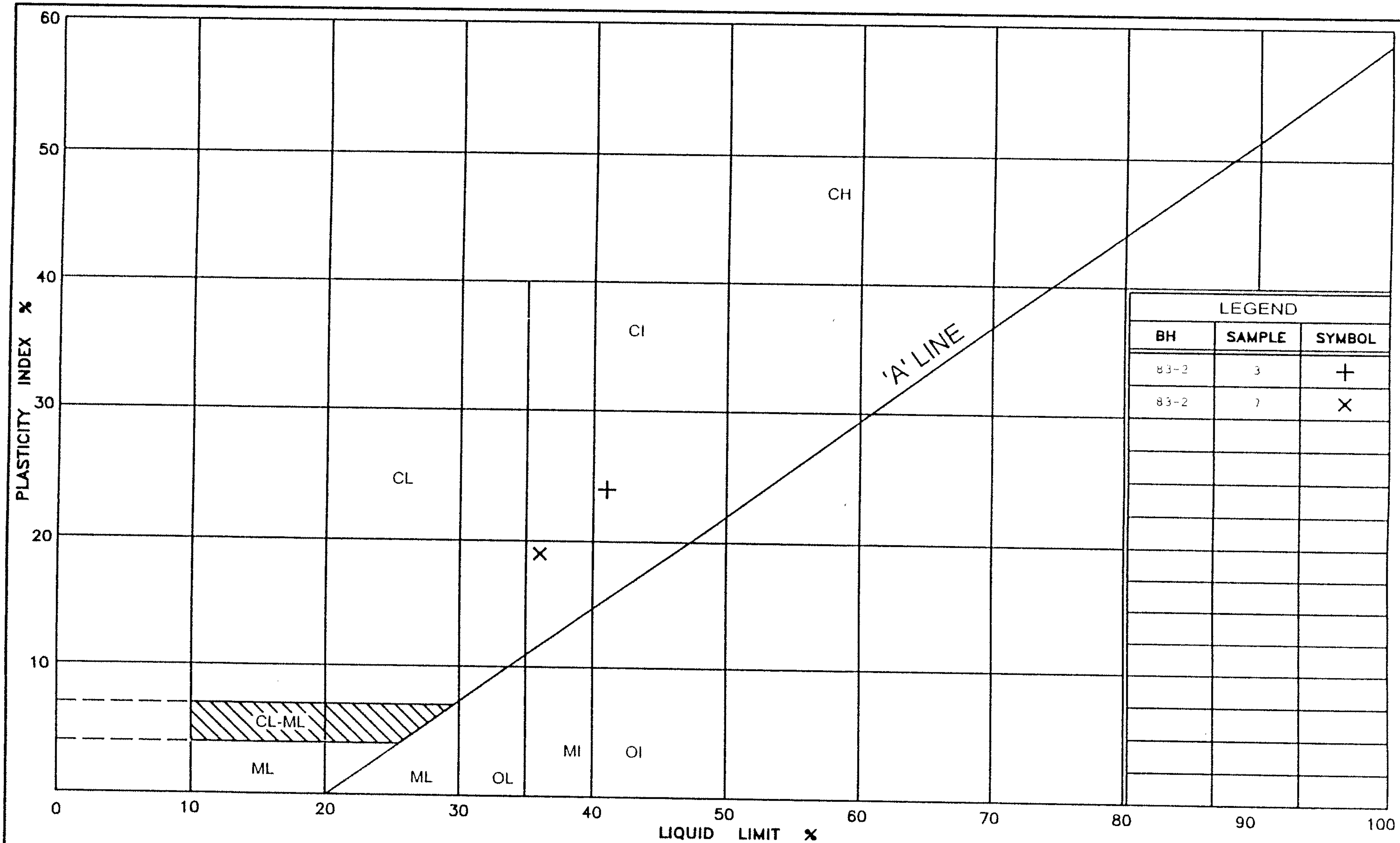
Brian R. Gray, M.Eng., P.Eng.
President

GD:lad

APPENDIX A

Figure 1 – Plasticity Chart

Figure 2 – Grain Size Distribution Chart



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 6-83

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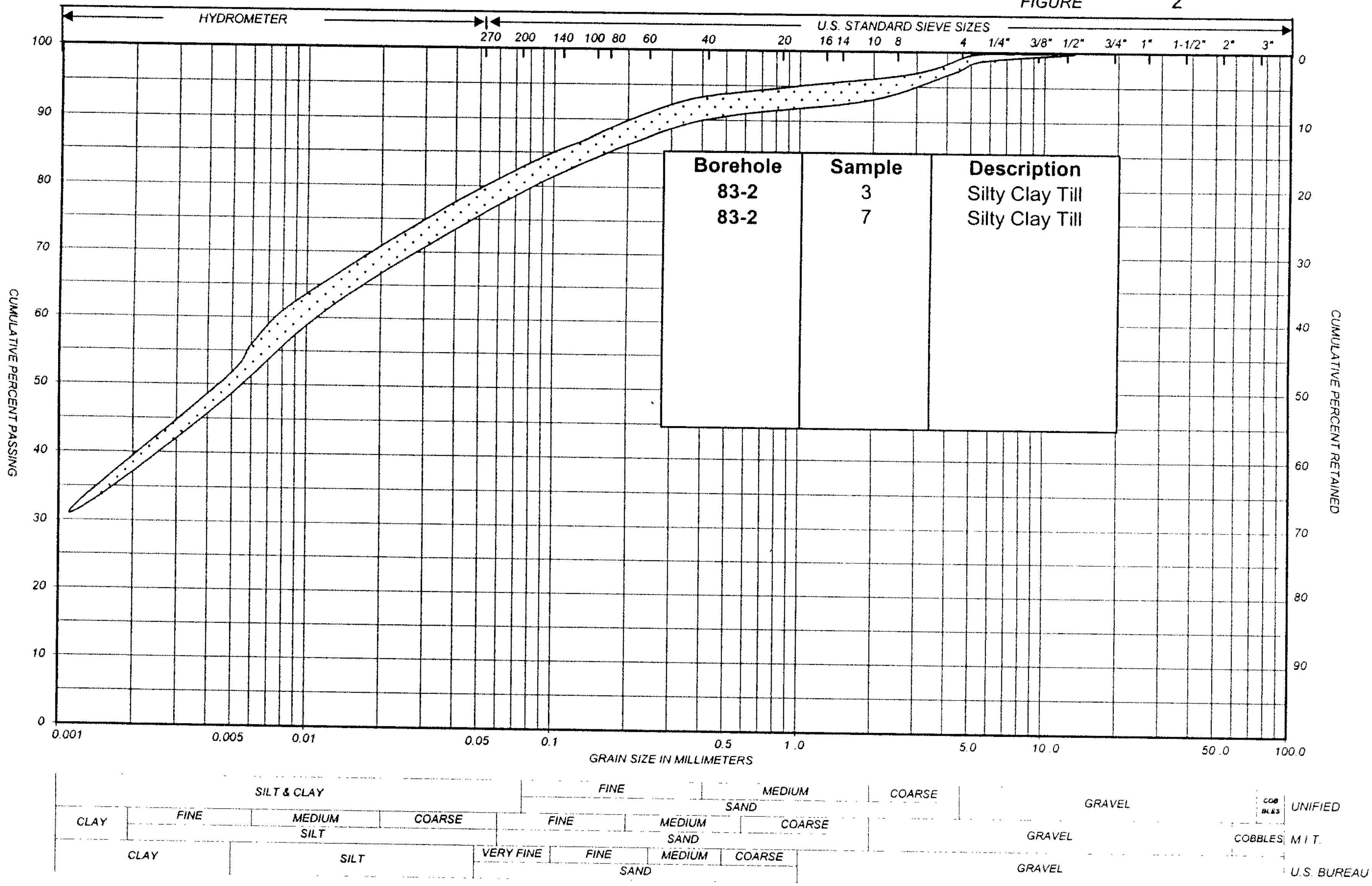
PML REF.	01TF073C
G.W.P.	60-00-00
FIGURE	2

PARTICLE SIZE DISTRIBUTION CHART



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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Foundation Design

RECORD OF BOREHOLE No 83-1

 METRIC

G.W.P. 60-00-00 LOCATION Co-ords. 4 617 717 N; 280 525 E. ORIGINATED BY MR

DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY GD

DATUM Geodetic DATE May 13, 2002 CHECKED BY MRA

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS
184.62	Ground Level					
0.00	Silty clay, trace to some sand, trace of gravel; Very Stiff; Brown (Fill)	1	SS	16		
183.22 / 1.40	Sandy clayey silt, trace of gravel; Very Stiff; Brown (Fill)	2	SS	21		
182.42 / 2.20	Silty clay, some sand, trace of gravel; Stiff to Hard; Brown (Till)	3	SS	10		
		4	SS	25		
		5	SS	42		
		6	SS	36		
	Very Stiff to Stiff; Grey	7	SS	21		
		8	SS	16		
		9	SS	14		
174.42 / 10.20	Silty clay, trace of sand, with numerous thin distorted lenses and partings of silt; Very Stiff; Grey	10	SS	25		
173.52 / 11.10	End of Borehole					

Borehole dry on completion of drilling

■ Penetrometer Test

Elevation scale: 184, 183, 182, 181, 180, 179, 178, 177, 176, 175, 174

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

Penetrometer values plotted: about 78 kPa at sample 7, about 38 kPa at samples 8 and 9, 162 at sample 10

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 83-2

METRIC

G.W.P. 60-00-00 LOCATION Co-ords. 4 677 718 N; 290 542 E. ORIGINATED BY MR

DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY GD

DATUM Geodetic DATE May 13, 2002 CHECKED BY MRA

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
182.70	Ground Level
0.00	Silty clay, some sand, trace of gravel Stiff to Hard Brown (Till) Very Stiff to Stiff Grey
175.60 / 7.10	Silt Compact Grey Saturated
173.10 / 9.60	End of Borehole

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	12	
2	SS	89	
3	SS	44	2 19 41 38
4	SS	41	
5	SS	16	
6	SS	14	
7	SS	9	2 15 44 39
8	SS	11	
9	SS	30	

ELEVATION SCALE: 182, 181, 180, 179, 178, 177, 176, 175, 174

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

GROUND WATER CONDITIONS

▼ Water level measured after drilling

■ Penetrometer Test

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

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

FORM DB-ML-112 REVISED 64-3040

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

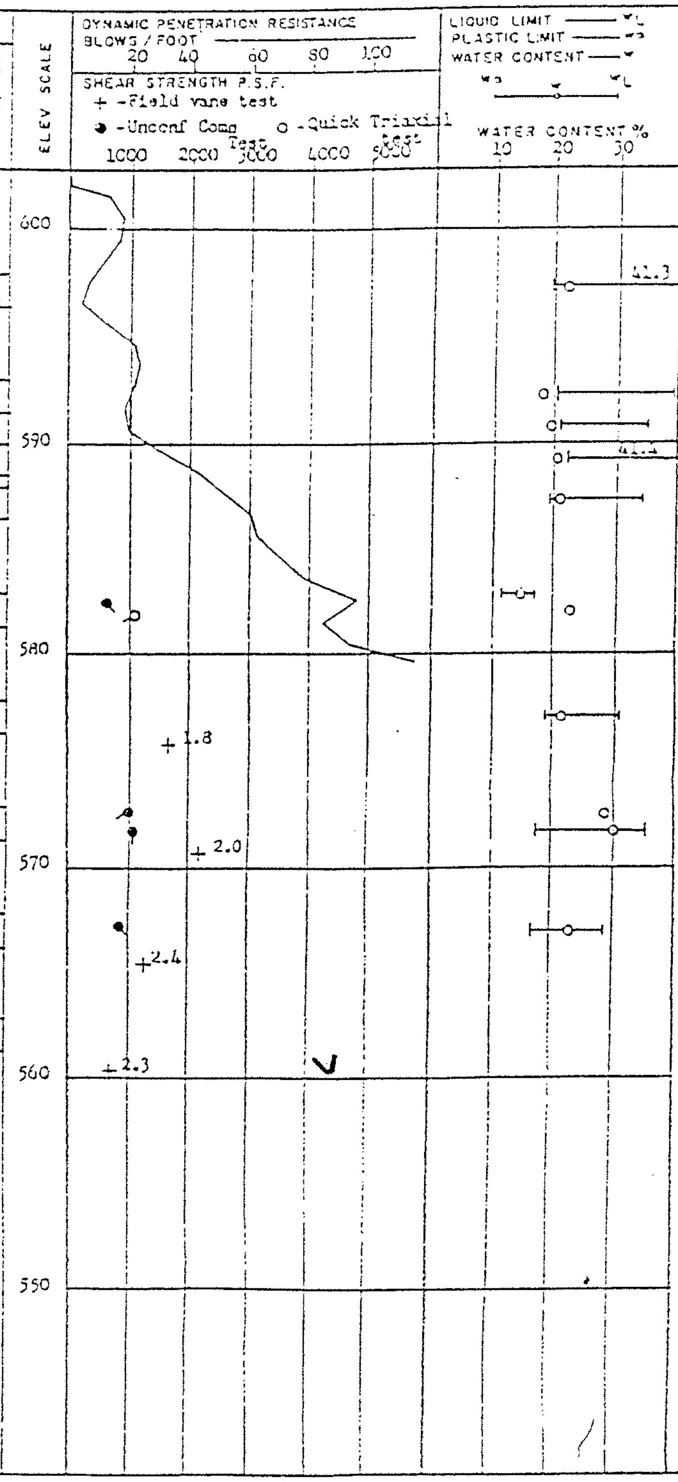
FOUNDATION SECTION

JOB 65-F-76 LOCATION Puce River Crossing of Hwy 401, Southeast of Bridge ORIGINATED BY L.P.

W.P. 693-64 BORING DATE July 15, 1965. COMPILED BY L.P.

DATUM Geodetic BOREHOLE TYPE Washboring CHECKED BY H.D.

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY P.C.F.	REMARKS
(183.73) 602.8 / 0.0	Silty clay, some sand, trace gravel (Glacial till) Stiff to hard, Grey-brown.	1	SS	11	128.5	Sa 27% Si 36% Cl 37%
		2	SS	12		
		3	SS	26	134	Sa 17% Si 43% Cl 40%
		4	SS	23		
(179.18) 587.8 / (4.55) 15.0	Clayey silt, some sand trace gravel, (Glacial till) stiff to firm, grey.	5	SS	17		
(177.58) 582.6 / (6.15) 20.2	Layer of sandy silt.	6	TW	PH	134	Sa&Gr 44% Si 35% Cl 21%
(176.58) 579.3 / (7.15) 23.5		7	SS	11		
		8	TW	PH	124 / 130	
		9	TW	PH	127	Sa 16% Si 48% Cl 36%
(171.23) 561.8 / (12.50) 41.0	6" Layer or pocket of silty sand	10	TW	PH		
(170.63) 559.8 / (13.10) 43.0	End of borehole.					



( ) Metric depths and elevations are bracketed.

FORM DH-MC-...REVISED

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 65-F-76
LOCATION Puce River Crossing of Hwy 401, Southwest of Bridge
ORIGINATED BY L.P.

W.P. 693-64
BORING DATE July 16, 1965.
COMPILED BY L.P.

DATUM Geodetic
BOREHOLE TYPE Washboring
CHECKED BY M.J.

	ELEV. / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. (Unconf Comp. Test)	WATER CONTENT %	BULK DENSITY P.C.F.	REMARKS
(183.18)	601.0	Ground Surface									
	0.0	Clayey silt to silty clay some sand, trace gravel (Glacial till) stiff to very stiff grey brown.					600				
				1	SS	17				136.5	
				2	SS	23				133	Sa15%Si 42% Cl 43%
				3	SS	17	590				
(179.53) (3.65)	589.0 / 12.0	Grey		4	TW	PH				129	Sa14%Si44% Cl 42%
				5	TW	PH		1.9		125	
(177.98) (5.20)	584.0 / 17.0	Silty sand - grey, dense.									Gr2%Sa73% Si&Cl 25%
				6	SS	37	580				
(176.63) (6.55)	579.5 / 21.5	End of borehole.									
							570				

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.: 1000 2000 3000 4000 5000

LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w

WATER CONTENT %: 10 20 30

( ) Metric depths and elevations are bracketed.

METRIC

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

CONT No
GWP No 60-00-00

HIGHWAY 401

PROPOSED BRIDGE WIDENING AT PUCE RIVER

BOREHOLE LOCATIONS & SOIL STRATA

SHEET

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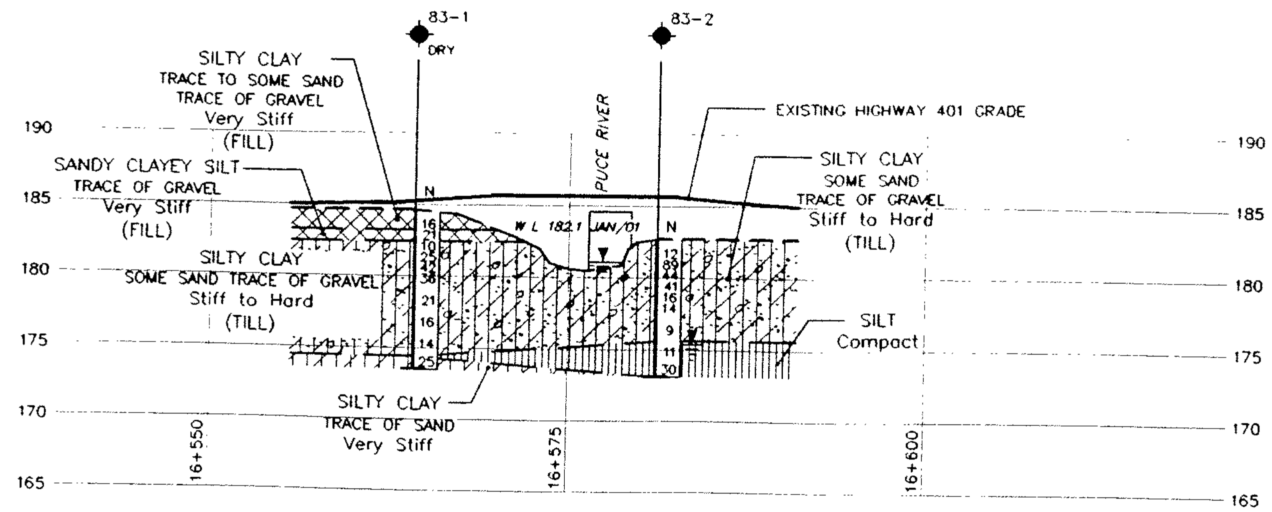
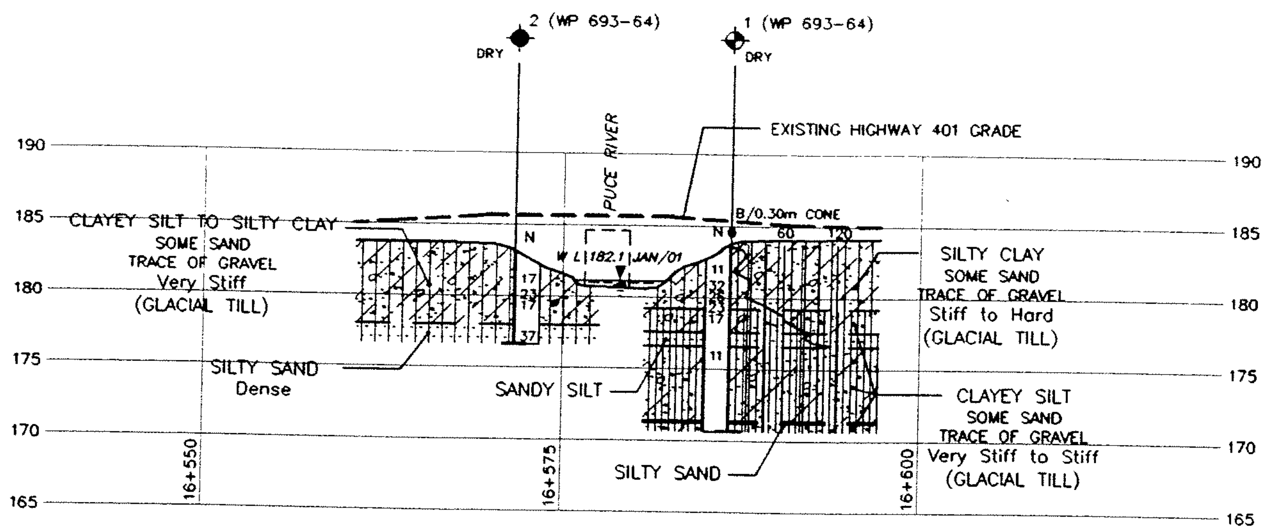
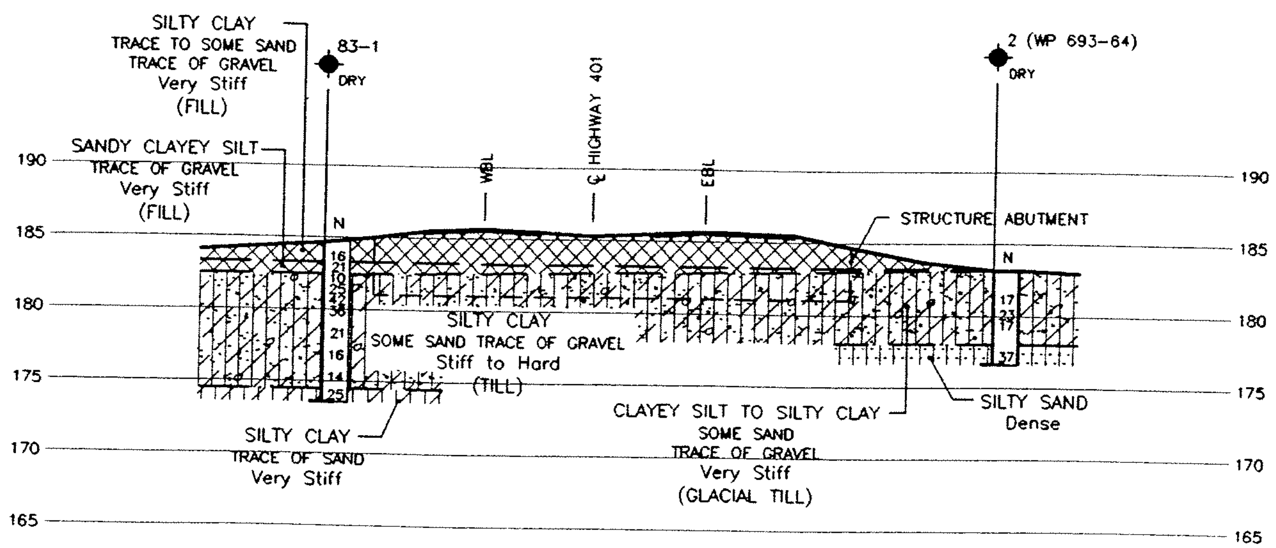
CONSULTING ENGINEERS



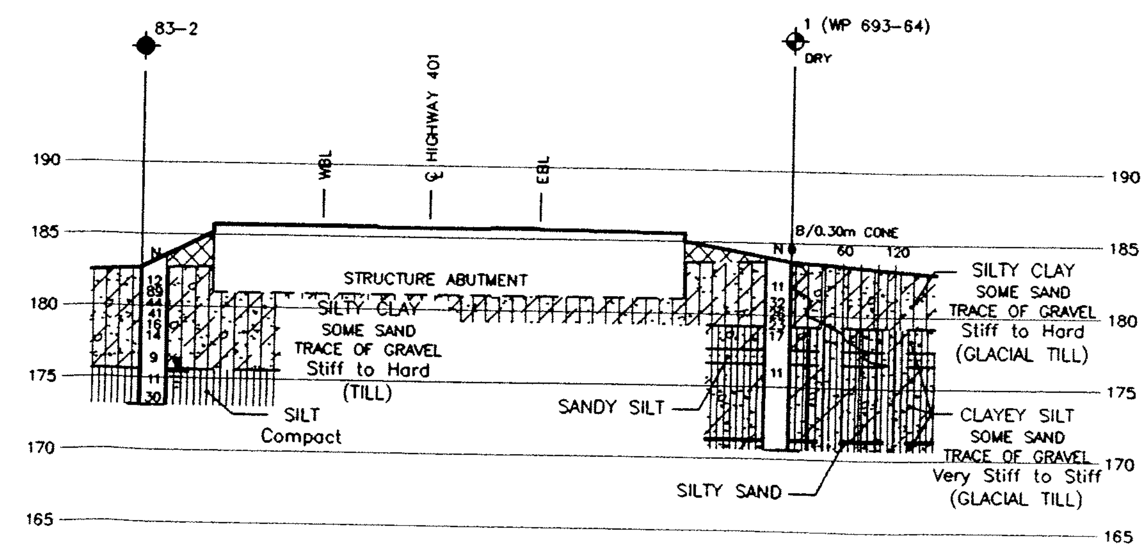
A-A



B-B



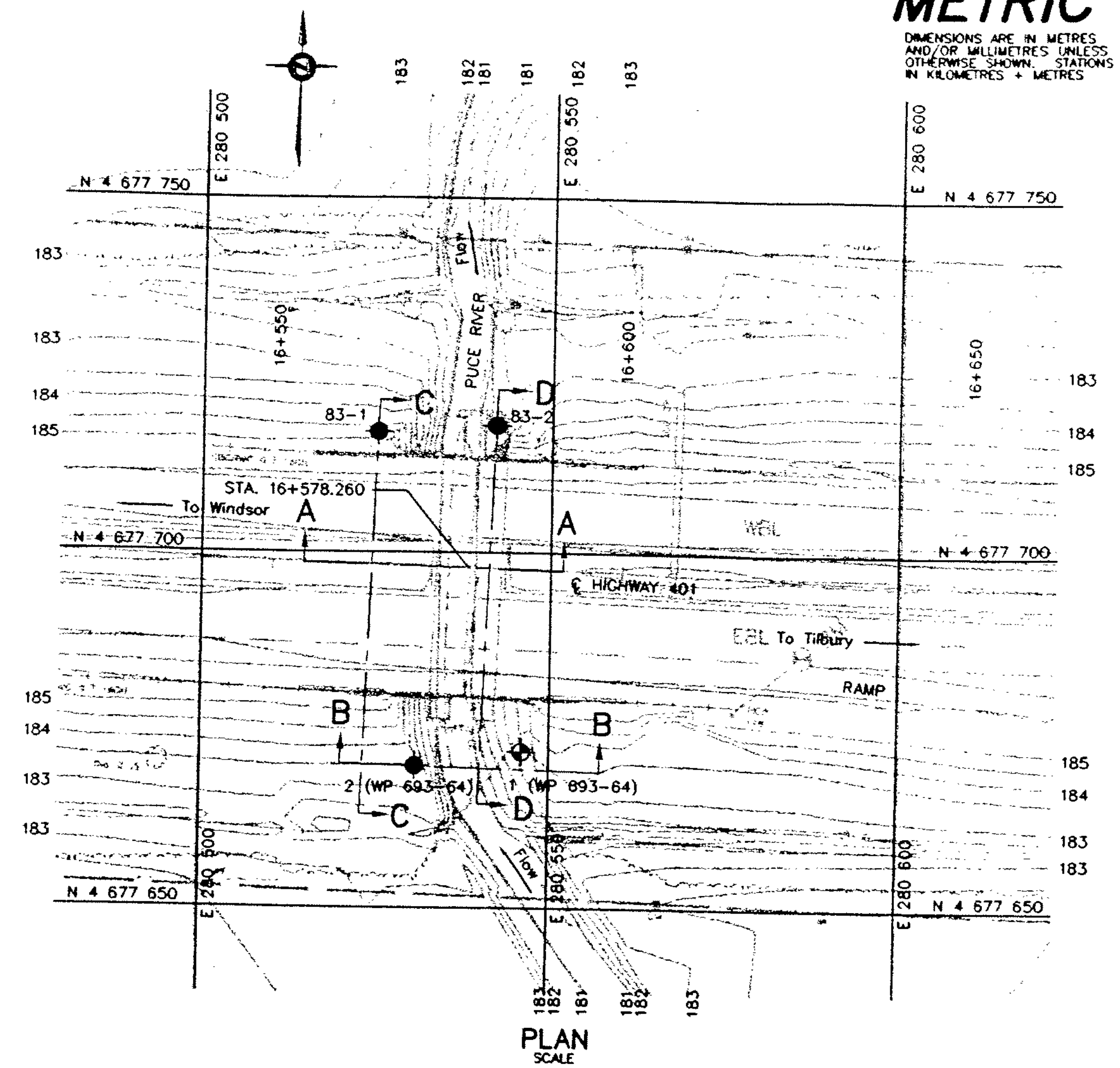
C-C



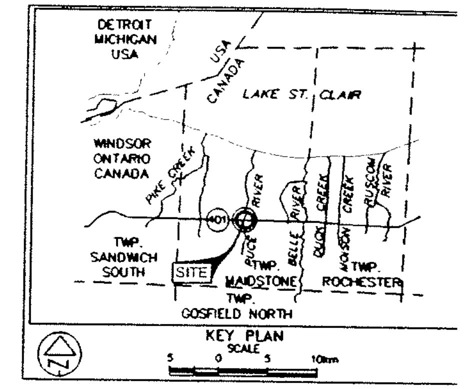
D-D

SECTIONS
SCALE
5 2.5 0 5 10m

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.



PLAN
SCALE
10 5 0 10 20m



KEY PLAN
SCALE
5 0 5 10km

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

BOREHOLES FOR CURRENT INVESTIGATION
GWP 60-00-00

BH No	ELEVATION	CO-ORDINATES NORTH	EAST
83-1	184.62	4 677 717	280 525
83-2	182.70	4 677 718	280 542

BOREHOLES FROM PREVIOUS INVESTIGATION
WP 693-64

BH No	ELEVATION	CO-ORDINATES NORTH	EAST
1 (WP 693-64)	183.73	4 677 672	280 546
2 (WP 693-64)	183.18	4 677 670	280 531

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

REF No Survey Plan 2001 Site # 6-83, entitled Proposed Crossing at Puce River and King's Highway 401, Prepared by Marshall Macklin Monaghan and

REVISIONS		
DATE	BY	DESCRIPTION

Geocres No.

HWY No 401 DIST 32

PetoMacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION DESIGN REPORT
FOR
WIDENING OF PUCE RIVER BRIDGE
G.W.P. 60-00-00, SITE 6-83
HIGHWAY 401
TOWN OF LAKESHORE, ONTARIO

Distribution:

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PML Ref: 01TF073C
Geocres No. 40J2-54

November 2002

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FOUNDATION DESIGN REPORT
for
Widening of Puce River Bridge
G.W.P. 60-00-00, Site 6-83
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 Puce 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 10 m (ref. drawing No. 401-MAI-E-01 'Proposed Crossing at Puce River. King's Highway 401 dated April 2001). Highway 401 will pass over Puce River at approximate Station 16+578, Highway 401 chainage, in the Town of Lakeshore (Township of Maidstone).

Road grade on Highway 401 at the bridge location is near elevation 185.5 (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 2.5 m. It is understood that the existing bridge is supported on shallow spread footings.

The subsurface stratigraphy revealed in the boreholes drilled at the site comprised a surficial fill, silty clay/clayey silt till and discontinuous deposits of silty clay, silt, sandy silt and silty sand.

FOUNDATIONS

Spread Footings

Supporting the bridge widenings on conventional spread footings founded in the native mineral soil is considered to be feasible at this site. Details concerning the design of the foundations supporting the existing bridge were not provided. Based on the river bed elevation shown on the drawing provided and the engineering properties of the soil revealed in the boreholes drilled at the site for this study, it is assumed that the footings are founded near elevation 181.0.

The bearing resistance of footings constructed on the very stiff to hard silty clay/clayey silt till at the inferred design founding level (assumed footing width of 2.5 m) is:

Factored Bearing Resistance at ULS	=	375 kPa
Bearing Resistance at SLS	=	250 kPa

Use of spread footings constructed on structural fill placed in the approach embankments does not appear to be suitable at this site since the embankment fill is approximately 2.5 m thick and a minimum 3.5 m thickness of engineered fill would be required to optimize the bearing resistance values at ULS and SLS.

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

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

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

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

Deep Foundations

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

ABUTMENT WALLS

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

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

where

K = lateral earth pressure coefficient

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

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 silty clay/clayey silt till overburden. The following parameters should be used in design of the system foundation:

	Granular "A"	Granular "B"	Silty Clay/Clayey Silt
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 should be responsible for design of the RSS (backfill, reinforcement, internal and external stability) and provide drawings to show pertinent information such as location, length, height, elevations, performance level, appearance etc.

APPROACH EMBANKMENTS

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

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

Consolidation settlements due to placing fill on the inorganic native 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 fill and native till. Based on the inferred design founding elevation of 181.0, the depth of excavation will vary from about 2 m beyond the toe of the embankment fill to as much as 4.5 m at the top of the fill. Cognizant of the fill present at the site, the 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.

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.

Use of steel sheeting to support the walls of the excavation is recommended due to the proximity of Puce River and potential "flood conditions" during construction.

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

Additional lateral resistance could be provided by installing tiebacks anchored in the very stiff to hard clay till. The factored pull-out resistance at ULS of soil anchors in the silty clay/clayey silt 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 predominantly very stiff silty clay/clayey silt 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 the water level in Puce River (elevation 182.1 in January 2001). The stabilized groundwater level, however, may vary in response to seasonal fluctuations and/or precipitation patterns.

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.

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

CLOSURE

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

Yours very truly

Peto MacCallum Ltd.

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Senior Foundation Engineer

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President

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APPENDICES

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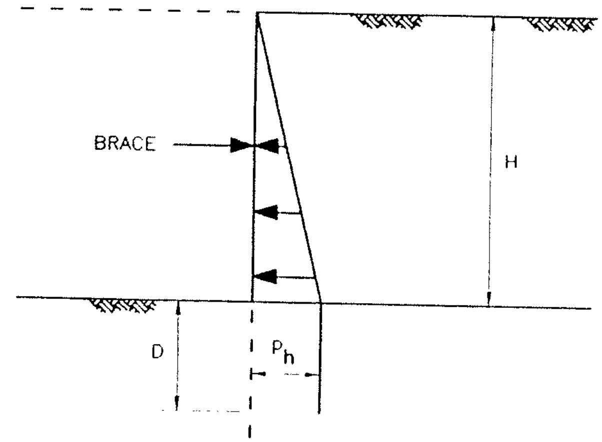
NOTES

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)

Peto MacCallum Ltd. CONSULTING ENGINEERS	LATERAL EARTH PRESSURE DISTRIBUTION SINGLY BRACED CUTS IN STIFF CLAYS	DATE	JOB NUMBER	FIGURE NO.
		Nov. 2002	01TF073C	1

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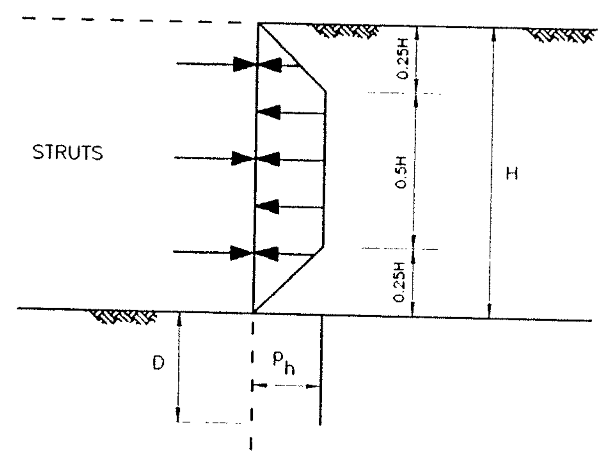
NOTES

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	01TF073C	2

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.

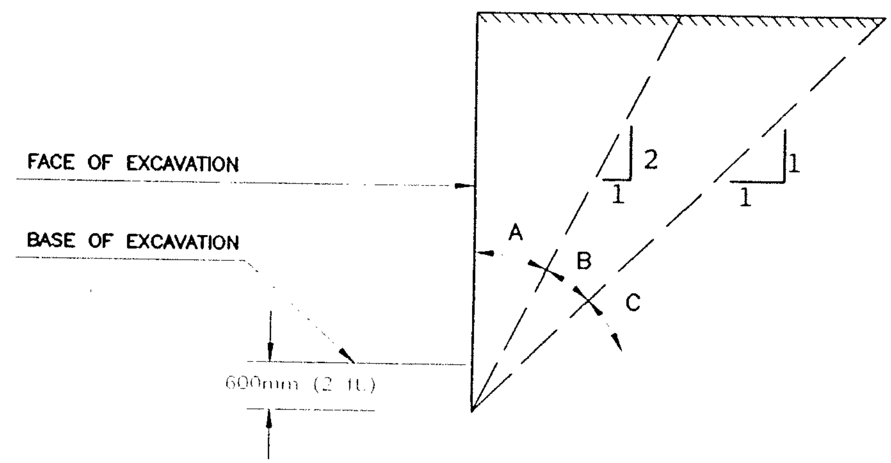
   Underpinning of foundations located in Zones A and B should extend at least into Zone C.

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

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

DATE	JOB NUMBER	FIGURE NO.
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