

**FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
BELLE RIVER ROAD OVERPASS
G.W.P. 60-00-00, SITE 6-85
HIGHWAY 401
TOWN OF LAKESHORE, ONTARIO**

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PML Ref: 01TF073E
Geocres No. 40J2-50

November 2002

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FOUNDATION INVESTIGATION REPORT

for
Belle River Road Overpass
G.W.P. 60-00-00, Site 6-85
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 overpass structure at Belle River 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 currently passes over Belle River Road at approximate Station 10+480, 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 Belle River Road. The structure to be widened carries Highway 401 traffic over Belle River Road. At the location of the structure, Highway 401 runs in the east-west direction.

The Highway 401 approaches comprise fill embankments of some 3 to 5 m in height. Belle River Road is cut 1 to 2 m below the adjacent ground surface. The difference in grade between the two roadways is approximately 6.2 m.

The site is located in the Township of Rochester in Essex County (Southwestern Ontario), some 25 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 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 January 14 to 16, 2002 and comprised four boreholes advanced to depths of 9.6 to 32.7 m, as summarized in the following table, at the locations indicated on Drawing 1 (Appendix B):

Location	Borehole No.	Depth, m
West Abutment, South Side	85-1	9.6
West Abutment, North Side	85-2	13.1
East Abutment, South Side	85-3	9.6
East Abutment, North Side	85-4	32.7

In addition, four boreholes (No. 221 to 224) drilled to depths of 5.0 to 6.6 m for the concurrent Embankment Foundation Investigation were used to supplement the subsurface data at the approach locations.

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: Top of round iron bar on south
side of Highway 401 west of
Belle River Road 18.848 RT
10+435.567
Elevation 187.184 (geodetic)

The boreholes were advanced using continuous flight hollow and solid stem augers, powered by truck-mounted and track-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 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. In situ vane shear and pocket penetrometer tests were also performed to further assess the shear strength of the cohesive soils.

The groundwater conditions in the boreholes were closely monitored during the course of the field work. Upon completion of drilling, a piezometer consisting of 19 mm PVC pipe slotted over the bottom 900 mm was installed in borehole 85-3 to monitor groundwater conditions. The annular space around the pipe was filled with filter sand and, after placing a bentonite seal, backfilled with auger cuttings to the ground surface as illustrated on the respective borehole log. The water level in the piezometer was measured on January 18, 21 and March 28, 2002.

Upon completion of drilling, the remaining boreholes (without piezometers) were backfilled with auger cuttings to the ground surface.

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

SUMMARIZED SUBSURFACE CONDITIONS

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

The borehole locations and stratigraphic profiles prepared from the borehole data are presented on Drawings 1 and 2.

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

Fill

Surficial fill was present in boreholes 85-1 to 85-4. The fill consisted of a 150 to 900 mm thick layer of fine to coarse sand and crushed gravel, underlain by 300 mm of gravel/cobbles over 900 mm of loose sand in borehole 85-1, by 650 mm of compact sand and silt in borehole 85-2, and by 900 mm of stiff clay in borehole 85-4.

It is noteworthy that the boreholes were drilled at the toe of the highway embankment fill. The embankment fill at this location is about 5 m high.

Topsoil

A 100 to 200 mm thick layer of topsoil was present surficially in boreholes 221, 223 and 224.

Silty Clay Till

Surficially in borehole 222 and directly beneath the fill/topsoil at depths of 0.1 to 2.1 m (elevation 179.5 to 184.5) in the remaining boreholes was cohesive silty clay till. The consistency of the clay till was stiff to hard in the upper portion of the unit, generally becoming firm to stiff with increasing depth. Standard penetration test N-values ranged from 10 to 23 in the upper 5 m of the structure holes, decreasing to 3 to 13 below this level, and from 14 to 57 in the approach holes. The results of vane shear testing carried out in this stratum at various depths indicate that the undisturbed and remolded shear strength values are in typical ranges of 80 to 150 kPa and 50 to 100 kPa respectively (soil sensitivity is about 1.5). A number of pocket penetrometer tests conducted within the unit gave the values of unconfined strength varying broadly between 15 and 200 kPa, decreasing with depth. (Values less than about 30 kPa were likely obtained from disturbed samples.)

The moisture content of the clay till typically ranged from 18 to 24%, increasing locally to 28 and 36%. The results of the Atterberg Limits tests are presented in Figure 1 (Appendix A). The clay till plots as a clay of intermediate plasticity, with liquid and plastic limits of 36 to 41 and 17 to 20, respectively. The results of particle size distribution analyses conducted on the clay till are presented in Figure 2 (Appendix A).

This unit had a confirmed thickness of 25 m in borehole 85-4 and was not penetrated upon termination of the remaining boreholes at depths of 5.0 to 13.1 m (elevation 168.4 to 179.7).

Silt

Underlying the clay till at a depth of 26.1 m (elevation 155.3) in borehole 85-4 was a 3.6 m thick deposit of cohesionless silt. The silt was compact in relative density (N-value of 10) and had a moisture content of 7%. It is worth noting that a cobble/boulder was encountered at the bottom of this deposit.

Sand and Silt Till

A 2 m thick layer of gravelly sand and silt till was revealed at a depth of 29.7 m (elevation 151.7). This deposit was cohesionless and very dense with a moisture content of 10%.

Silt Till

Clayey sandy silt till was identified below the sand and silt till at a depth of 31.7 m (elevation 149.7). This till unit was hard/very dense in consistency/relative density and had a moisture content of 21%. Borehole 85-4 was terminated within this deposit at a depth of 32.7 m (elevation 148.7).

Groundwater

Water was observed in three boreholes during or upon completion of drilling. During the field work, perched water was detected in the surficial layer of sand and gravel in borehole 85-1. Water was observed in borehole 85-4 at 12.7 m depth (elevation 168.7) during drilling and at 6.4 m depth (elevation 175.0) one day later. Free water was not observed in boreholes 85-2, 221 to 224 during the course of the field work.

A piezometer was installed in borehole 85-3. Three sets of piezometer readings taken on January 18, 21 and March 28, 2002 showed water levels to be at depths of 9.0, 8.9 and 0.3 m (elevation 172.7, 172.8 and 181.4 respectively).

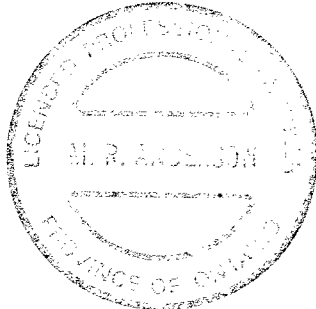
CLOSURE

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. It was reviewed by Mr. M.R. Anderson, M.Eng., P.Eng., Senior Foundation Engineer, and 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.



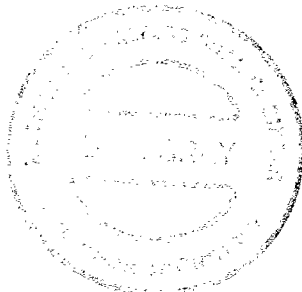
A handwritten signature of Murray R. Anderson in black ink.

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



A handwritten signature of Dennis W. Kerr in black ink.

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



A handwritten signature of Brian R. Gray in black ink.

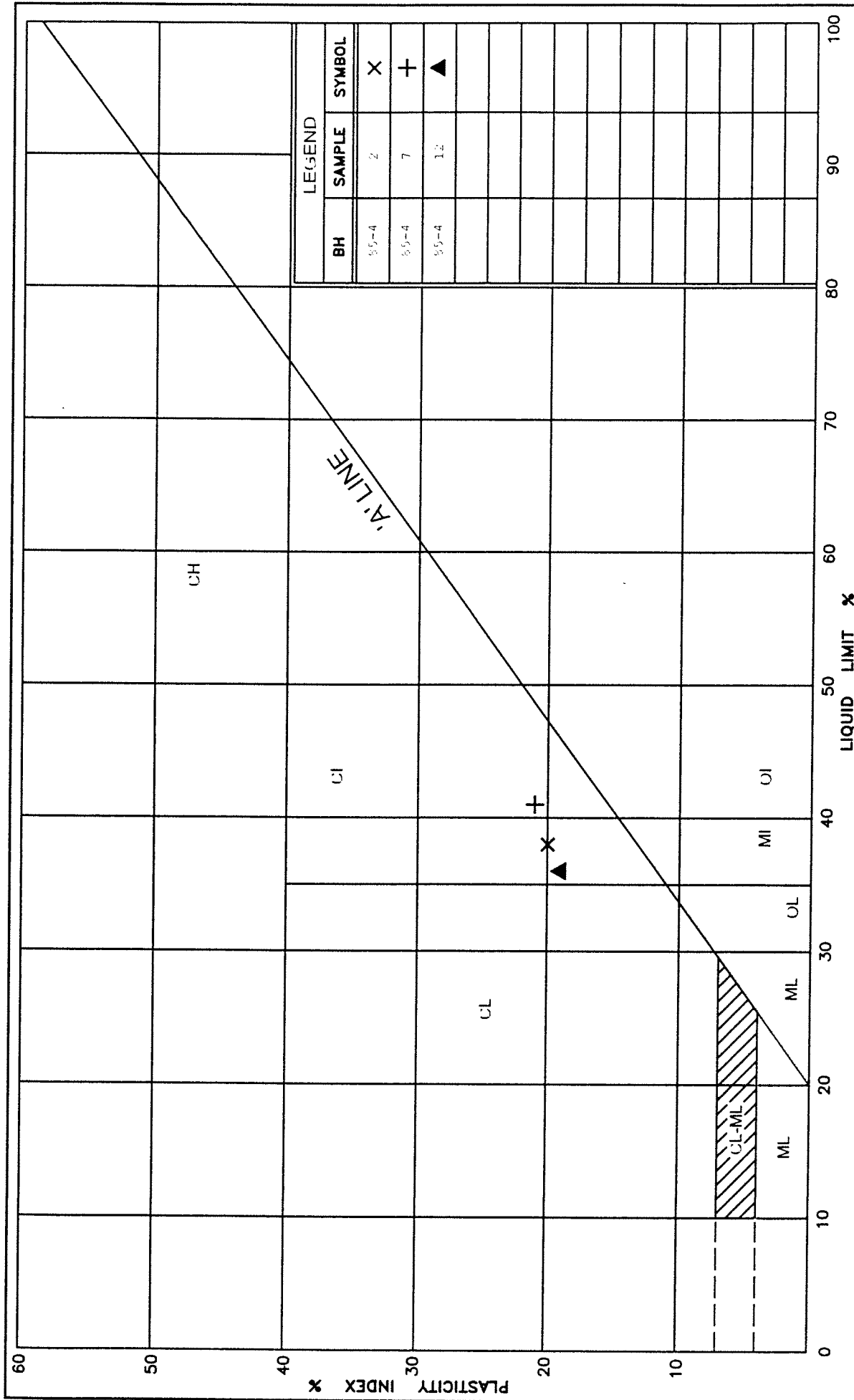
Brian R. Gray, M.Eng., P.Eng.
President

GD:lad

APPENDIX A

Figure 1 – Plasticity Chart

Figure 2 – Grain Size Distribution Chart



PLASTICITY CHART

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

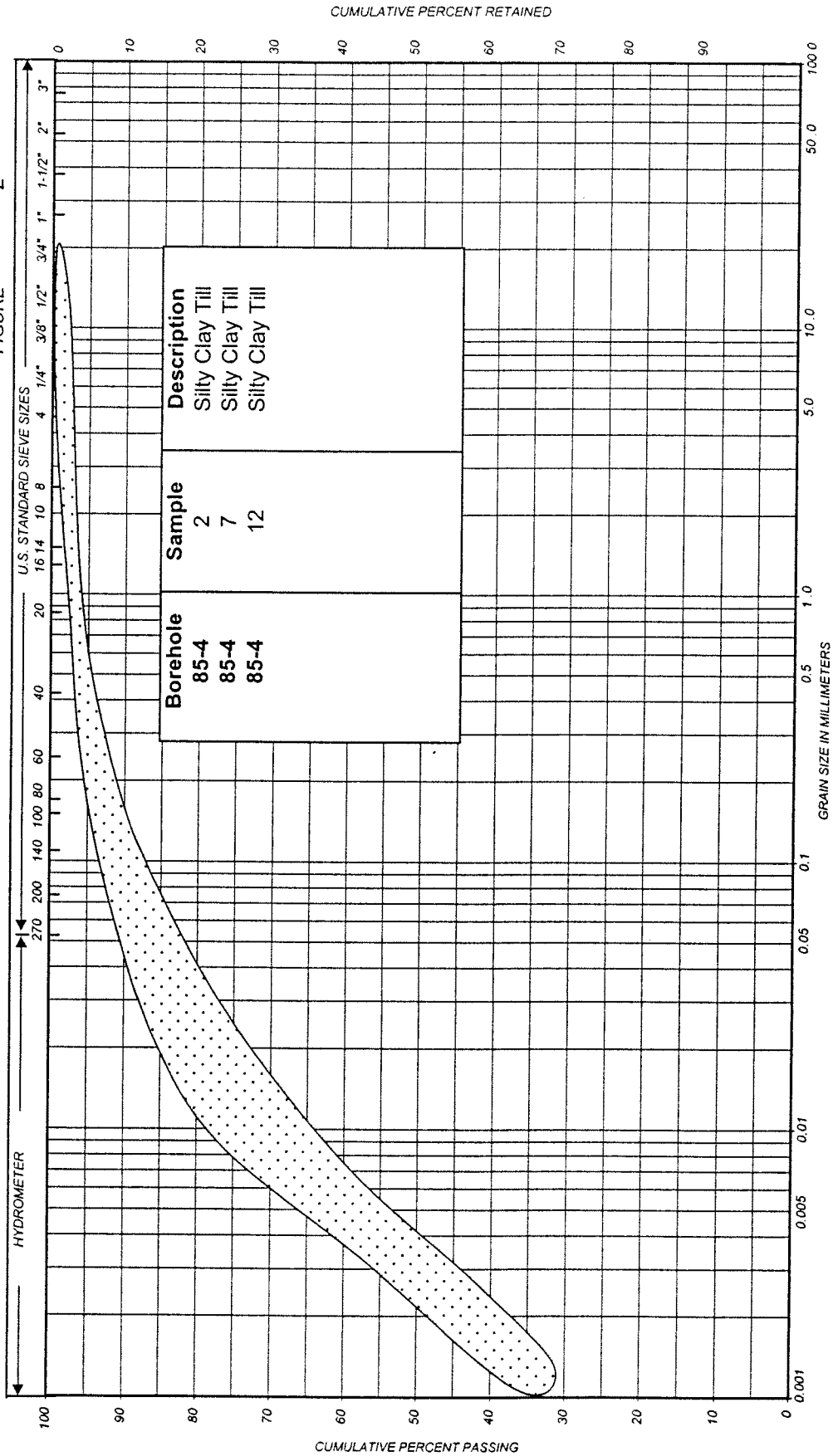
FIG No 1

HIGHWAY 401

G.W.P. 60-00-00, Site 6-35

PML REF. 01TF073E
G>W>P> 60-00-00
FIGURE 2

PARTICLE SIZE DISTRIBUTION CHART



U.S. BUREAU									
CLAY		SILT & CLAY		FINE		COARSE		GRAVEL	
CLAY		FINE		MEDIUM		SAND		COBBLES	
CLAY		FINE		MEDIUM		SAND		COBBLES	
CLAY		FINE		MEDIUM		SAND		COBBLES	
CLAY		FINE		MEDIUM		SAND		COBBLES	
CLAY		FINE		MEDIUM		SAND		COBBLES	
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CLAY		FINE		MEDIUM		SAND		COBBLES	
CLAY		FINE		MEDIUM		SAND		COBBLES	

APPENDIX B

Record of Borehole Sheets

Drawings 1 and 2

LIST OF ABBREVIATIONS

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

<u>CONSISTENCY</u>	<u>'N' BLOWS/0.3 m</u>	<u>c kPa</u>	<u>DENSENESS</u>	<u>'N' BLOWS/0.3 m</u>
VERY SOFT	0 - 2	0 - 12	VERY LOOSE	0 - 4
SOFT	2 - 4	12 - 25	LOOSE	4 - 10
FIRM	4 - 8	25 - 50	COMPACT	10 - 30
STIFF	8 - 15	50 - 100	DENSE	30 - 50
VERY STIFF	15 - 30	100 - 200	VERY DENSE	> 50
HARD	> 30	> 200		

W.T.P.L. WETTER THAN PLASTIC LIMIT D.T.P.L. DRIER THAN PLASTIC LIMIT
A.P.L. ABOUT PLASTIC LIMIT

TYPE OF SAMPLE

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

SOIL TESTS

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

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

■ - UNDRAINED SHEAR STRENGTH DETERMINED FROM POCKET PENETROMETER TEST.

RECORD OF BOREHOLE No 85-1

1 of 1 METRIC

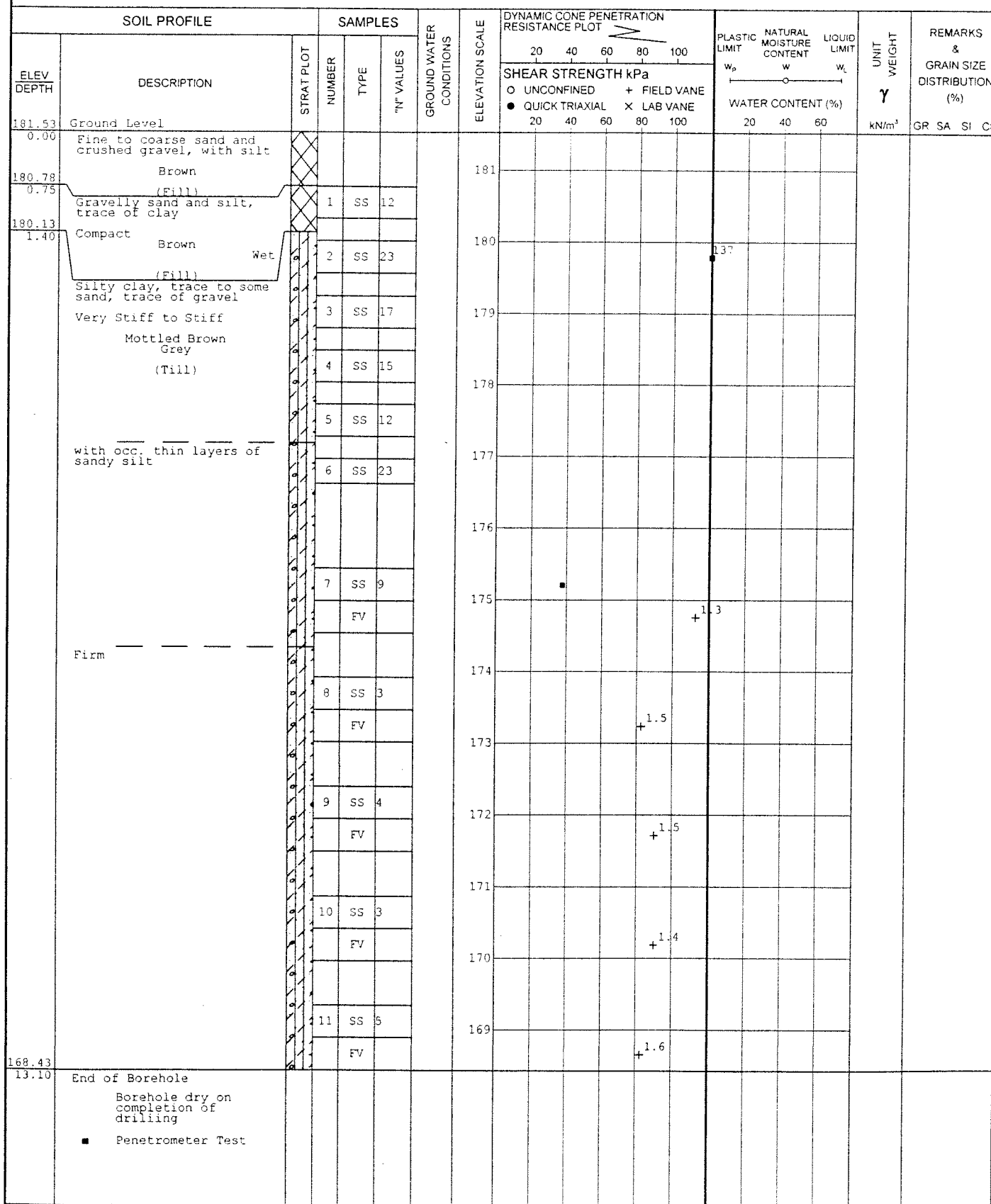
G.W.P. 60-00-00 LOCATION 4 677 465 N; 287 241 E. ORIGINATED BY MR
DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY GD
DATUM Geodetic DATE January 15, 2002 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100	20 40 60 80 100	w _p	w	w _L		
						○ UNCONFINED	+ FIELD VANE							
						● QUICK TRIAXIAL	x LAB VANE							
181.62	Ground Level													
0.00	Fine to coarse sand and crushed gravel, with silt													
180.72	Brown (Fill)						181							
0.90	Gravel to cobbles, with sand		1	SS	45									
180.42	Brown (Fill)		2	SS	5		180							
1.20	Fine to coarse sand, some silt													
179.52	Loose		3	SS	14		179							
2.10	Brown Saturated (Fill)		4	SS	22		178							
	Silty clay, trace to some sand, trace of gravel		5	SS	16		177							
	Stiff to Very Stiff		6	SS	15/20cm*		176							
	Grey (Till)		7	SS	6		175							
	Firm			FV			174							
			8	SS	7		173							
				FV										
172.02	End of Borehole 2002-1-15		9	SS	9									
9.60	Water level observed during drilling													
	Penetrometer Test													

RECORD OF BOREHOLE No 85-2

1 of 1 METRIC

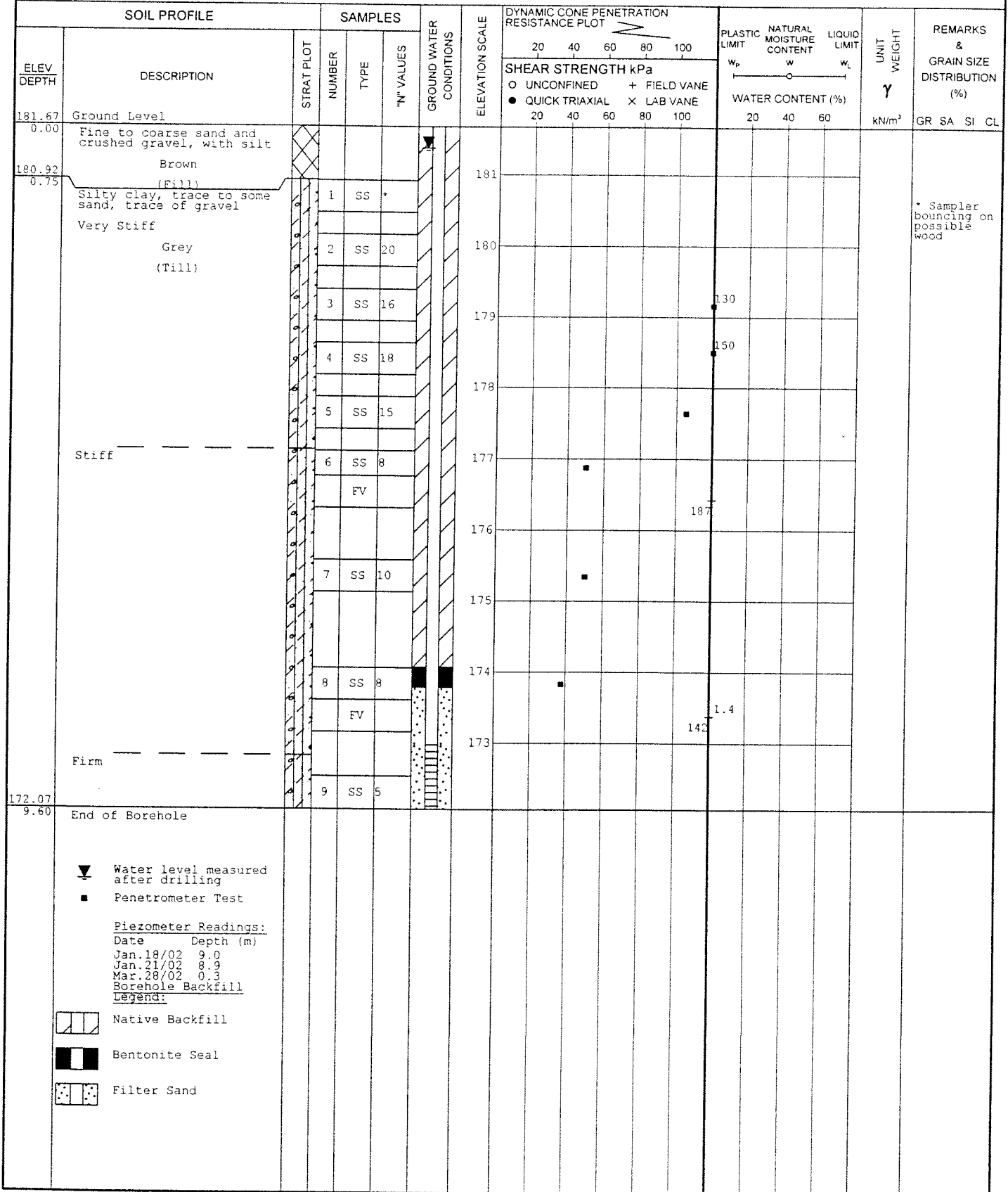
G.W.P. 60-00-00 LOCATION 4 677 523 N; 397 349 E. ORIGINATED BY MR
DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY GD
DATUM Geodetic DATE January 15, 2002 CHECKED BY MRA



RECORD OF BOREHOLE No 85-3

1 of 1 METRIC

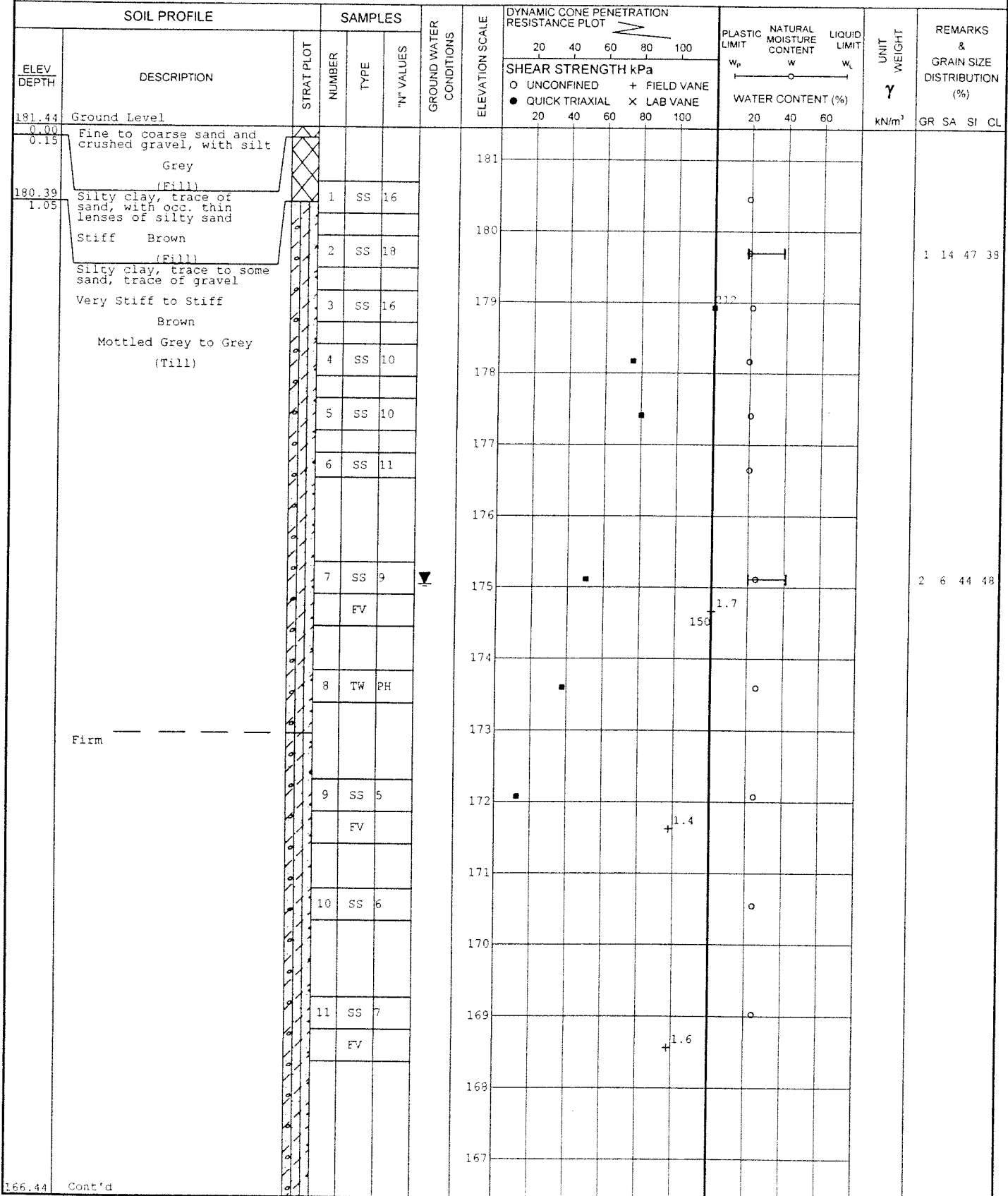
G.W.P. 60-00-00 LOCATION 4 677 465 N; 287 257 E. ORIGINATED BY MR
DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY GD
DATUM Geodetic DATE January 16, 2002 CHECKED BY MRA



RECORD OF BOREHOLE No 85-4

1 of 3 METRIC

G.W.P. 60-00-00 LOCATION 4 677 522 N; 287 261 E. ORIGINATED BY MR
DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY GD
DATUM Geodetic DATE January 14, 2002 CHECKED BY MRA



RECORD OF BOREHOLE No 85-4

2 of 3 METRIC

G.W.P. 60-00-00 LOCATION 4 677 522 N; 287 261 E. ORIGINATED BY MR
DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY GD
DATUM Geodetic DATE January 14, 2002 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
181.44	Ground Level													
15.00	Silty clay, trace to some sand, trace of gravel		12	SS	6									1 13 46 40
	Firm													
	Grey (Till)													
			13	SS	4									
	Stiff													
			14	SS	13									
			15	SS	8									
155.34	Silt													
26.10	Compact													
	Grey													
	Wet													
			16	SS	10									
151.74	cobble													
29.70	Cont'd													

RECORD OF BOREHOLE No 85-4

3 of 3 METRIC

G.W.P. 60-00-00 LOCATION 4 677 522 N; 287 261 E. ORIGINATED BY MR
DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY GD
DATUM Geodetic DATE January 14, 2002 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100					
181.44	Ground Level												
	Gravelly sand and silt Very dense Grey (Till) (Cont'd)		17	SS	92/23cm		151						
149.74							150						
31.70	Clayey sandy silt, some gravel Hard/Very Dense Grey (Till)												
148.74			18	SS	60/10cm		149						
32.70	End of Borehole												
	■ Penetrometer Test												
	▼ Water level measured after drilling												

RECORD OF BOREHOLE No 221

1 of 1 METRIC

G.W.P. 60-00-00 LOCATION Co-ords. 4 677 526 N; 287 218 E. ORIGINATED BY MR
DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY GD
DATUM Geodetic DATE March 19, 2002 CHECKED BY MPA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						
183.34	Ground Level							20	40	60	80	100				
0.00	Topsoil, silty clay															
0.15	Dark Brown Silty clay, some sand, trace of gravel						183									
	Hard		1	SS	39											
	Brown (Till)		2	SS	47		182									
							181									
			3	SS	34		180									
	trace to some sand						179									
	Stiff Grey		4	SS	14											
							178									
176.79			5	SS	14		177									
6.55	End of Borehole															
	Borehole dry on completion of drilling															

RECORD OF BOREHOLE No 222

1 of 1 METRIC

G.W.P. 60-00-00 LOCATION Co-ords. 4 677 464 N; 287 312 E. ORIGINATED BY MR
DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY JD
DATUM Geodetic DATE March 21, 2002 CHECKED BY NRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40					
182.94	Ground Level													
0.00	Silty clay, trace to some sand, trace of gravel													
	Hard													
	Brown (Till)		1	SS	46									
			2	SS	45									
	Very Stiff Grey		3	SS	26									
			4	SS	16									
176.39	End of Borehole		5	SS	15									
6.55	Borehole dry on completion of drilling													
	■ Penetrometer Test													

RECORD OF BOREHOLE No 223

1 of 1 METRIC

G.W.P. 60-00-00 LOCATION Co-ords. 4 677 521 N; 287 291 E. ORIGINATED BY MR
DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY GD
DATUM Geodetic DATE March 21, 2002 CHECKED BY MRA

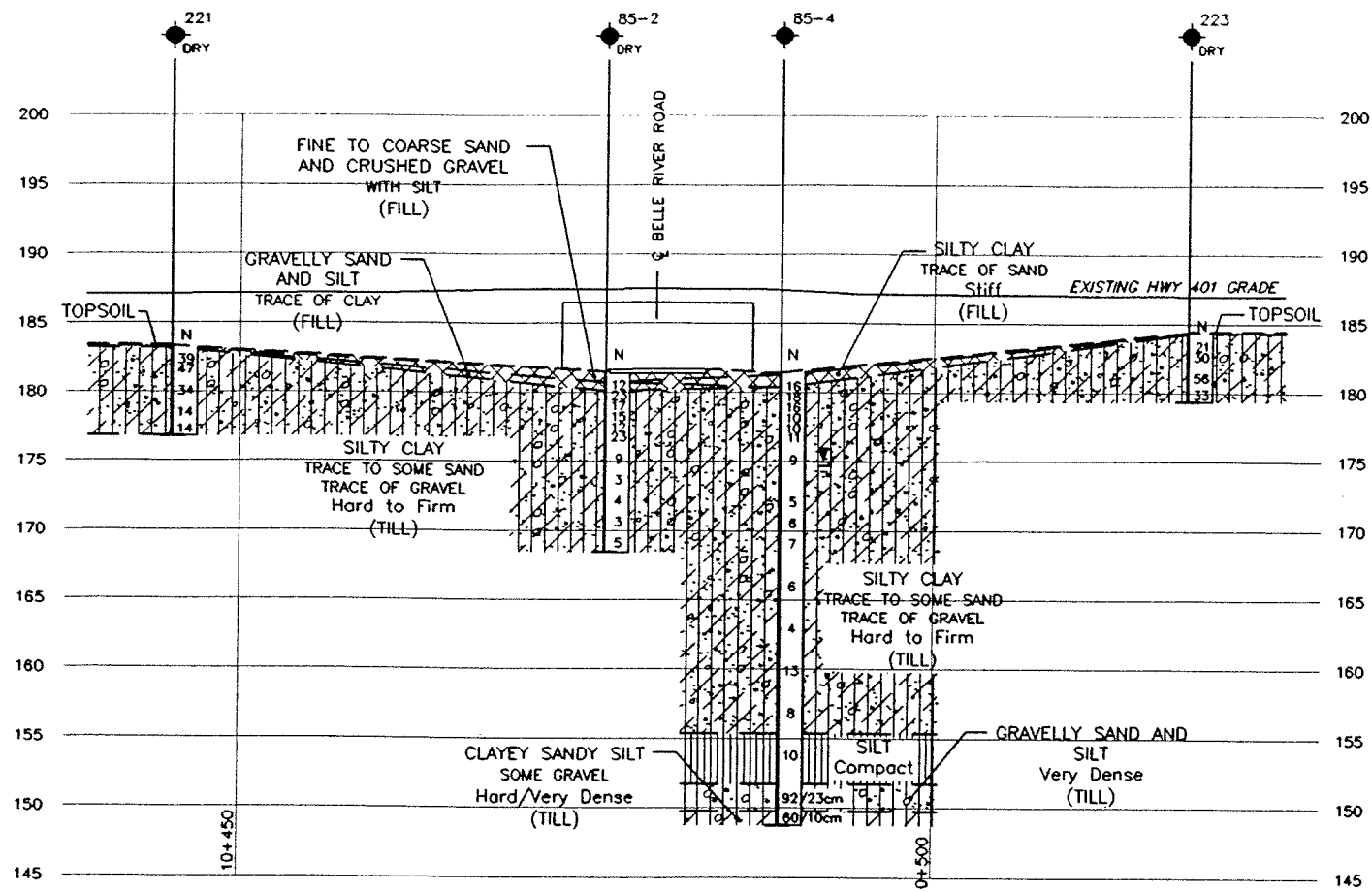
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
184.39	Ground Level													
0.00	Topsoil													
0.10	Silty clay, trace to some sand, trace of gravel													
	Very Stiff Brown (Till)		1	SS	21		184							
	Hard		2	SS	30		183							
							182							
			3	SS	56		181							
	Mottled Grey						180							
179.34			4	SS	33									
5.05	End of Borehole													
	Borehole dry on completion of drilling													

RECORD OF BOREHOLE No 224

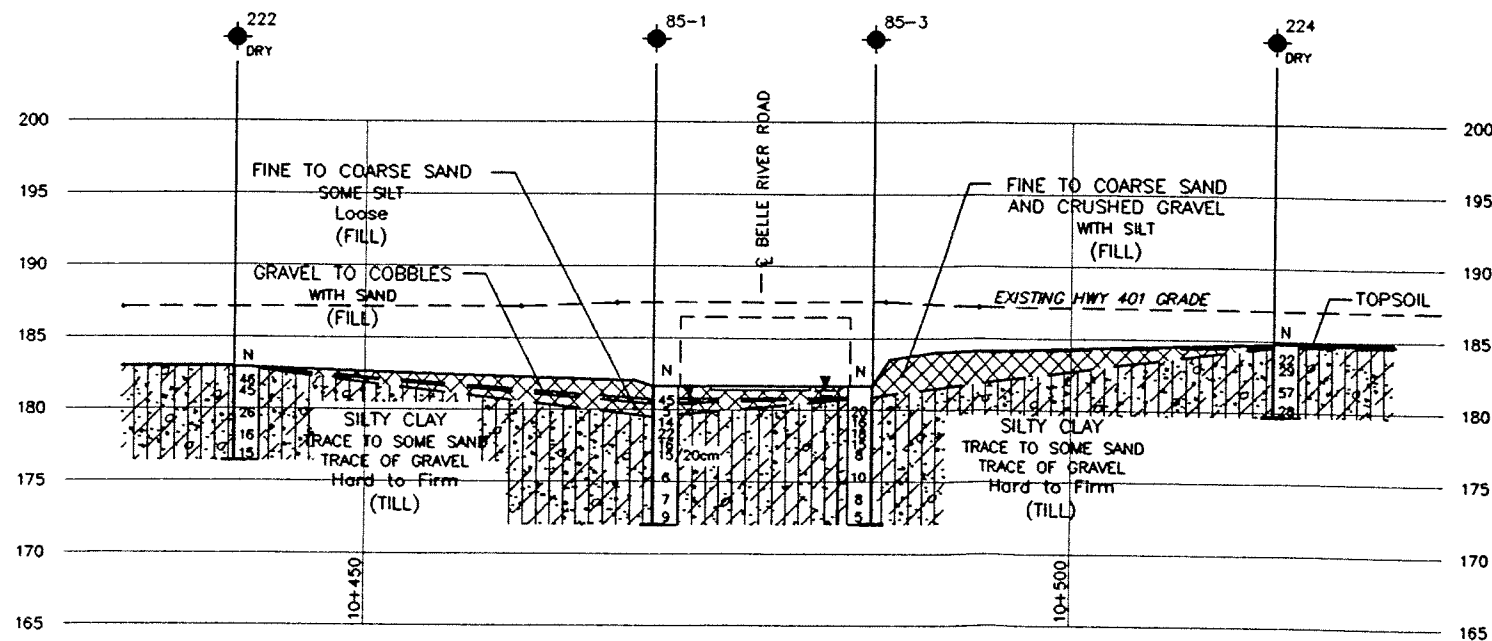
1 of 1 METRIC

G.W.P. 60-00-00 LOCATION Co-ords. 4 677 463 N; 287 285 E. ORIGINATED BY MR
DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY GD
DATUM Geodetic DATE March 21, 2002 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
184.74	Ground Level													
0.00	Topsoil													
0.20	Silty clay, trace to some sand, trace of gravel													
	Very Stiff Brown (Till)		1	SS	22		184							
			2	SS	29		183							
	Hard						182							
			3	SS	57		181							
	Very Stiff Grey						180							
179.69			4	SS	28									
5.05	End of Borehole													
	Borehole dry on completion of drilling													



A-A



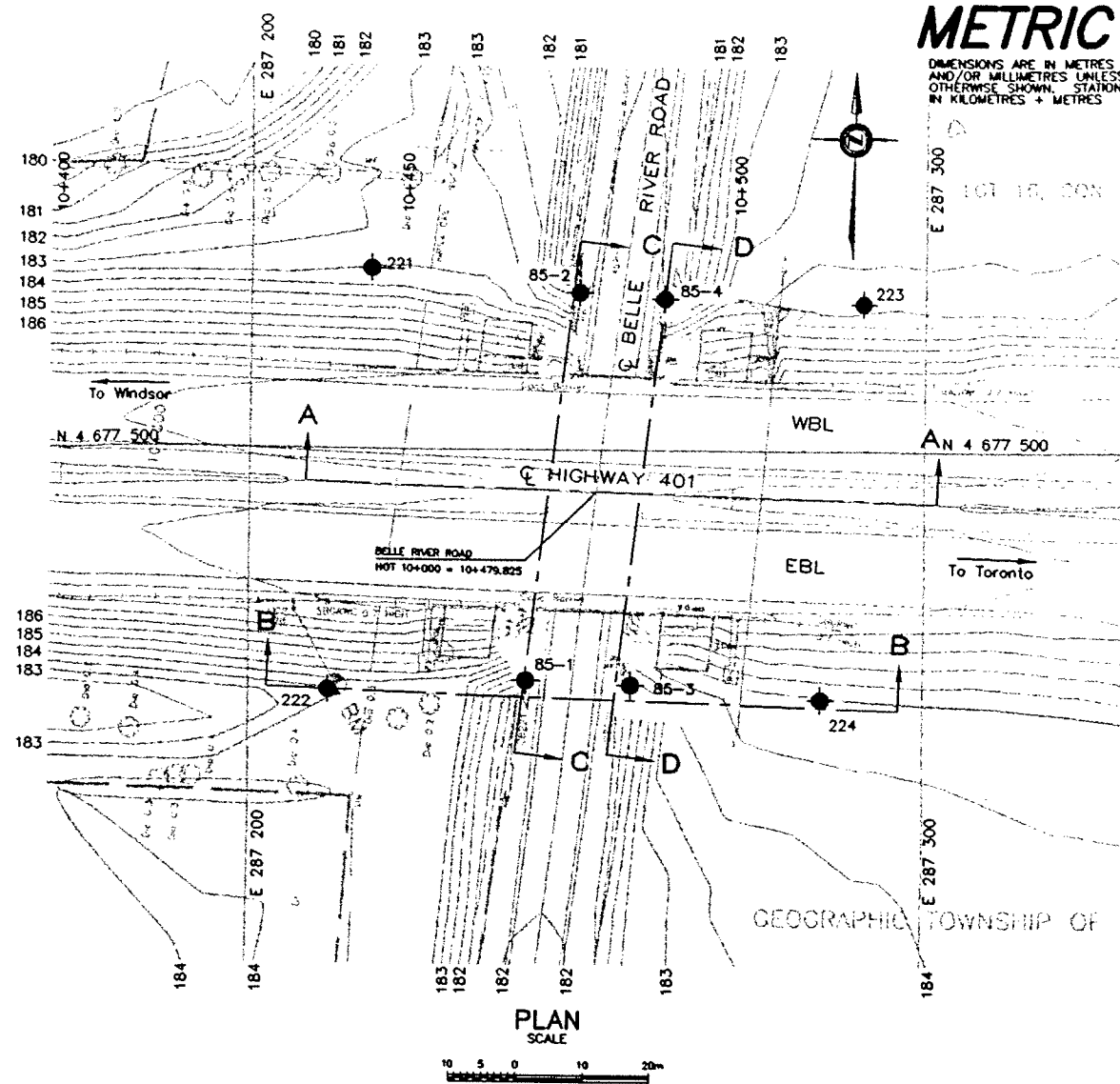
B-B

SECTIONS

SCALE
5 2.5 0 5 10m

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.



PLAN

SCALE
10 5 0 10 20m

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

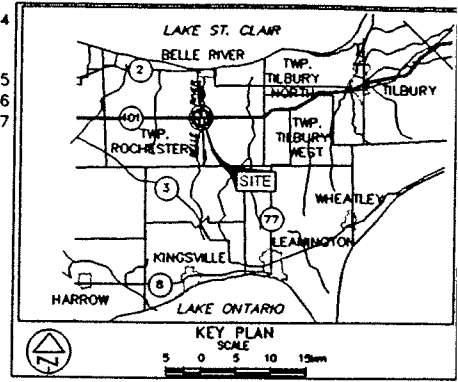
CONT No
GWP No 60-00-00

PROPOSED OVERPASS AT BELLE RIVER 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 - Mar 2002
- Head
- ARTESIAN WATER Encountered
- PIEZOMETER

BH No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
85-1	181.62	4 677 465	287 241
85-2	181.53	4 677 523	287 249
85-3	181.67	4 677 465	287 257
85-4	181.44	4 677 522	287 261
221	183.34	4 677 526	287 218
222	182.94	4 677 464	287 212
223	184.39	4 677 521	287 291
224	184.74	4 677 463	287 285

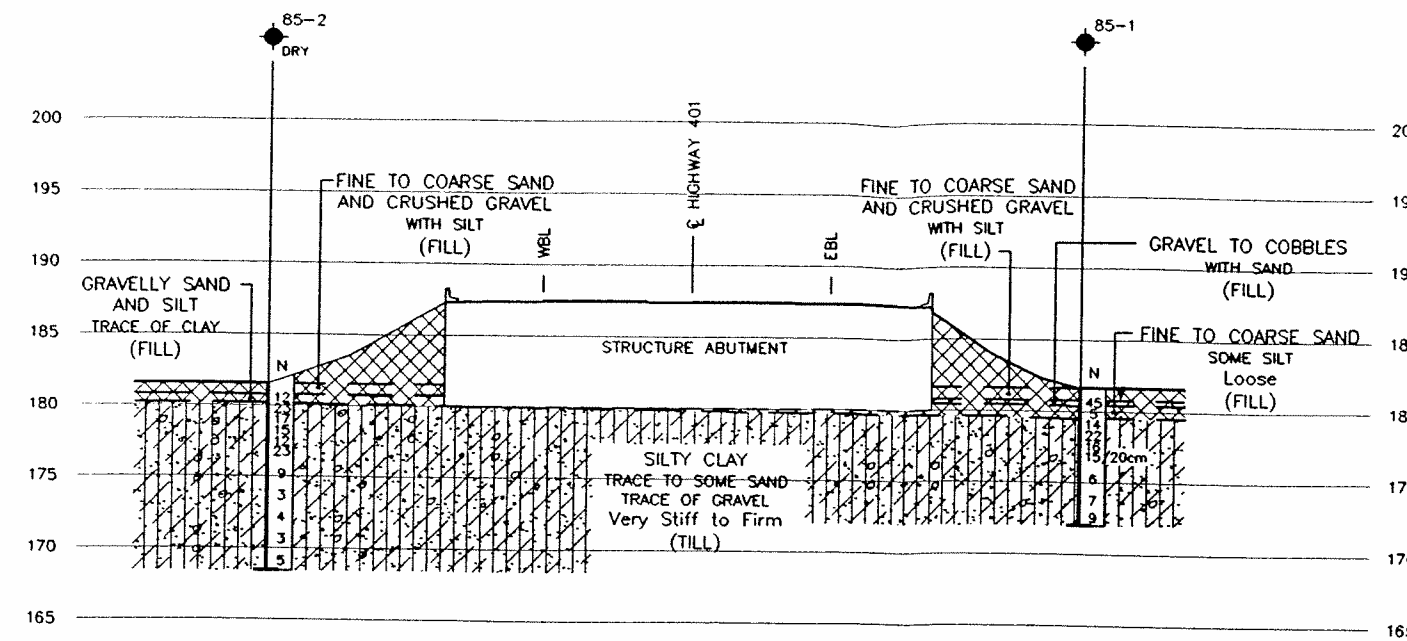
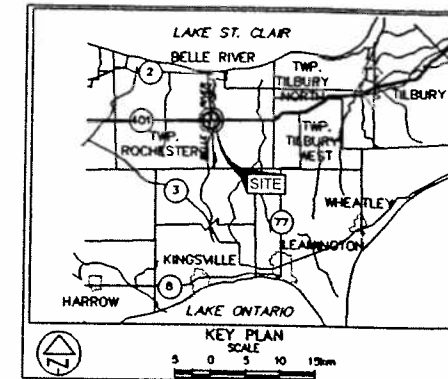
NOTE

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

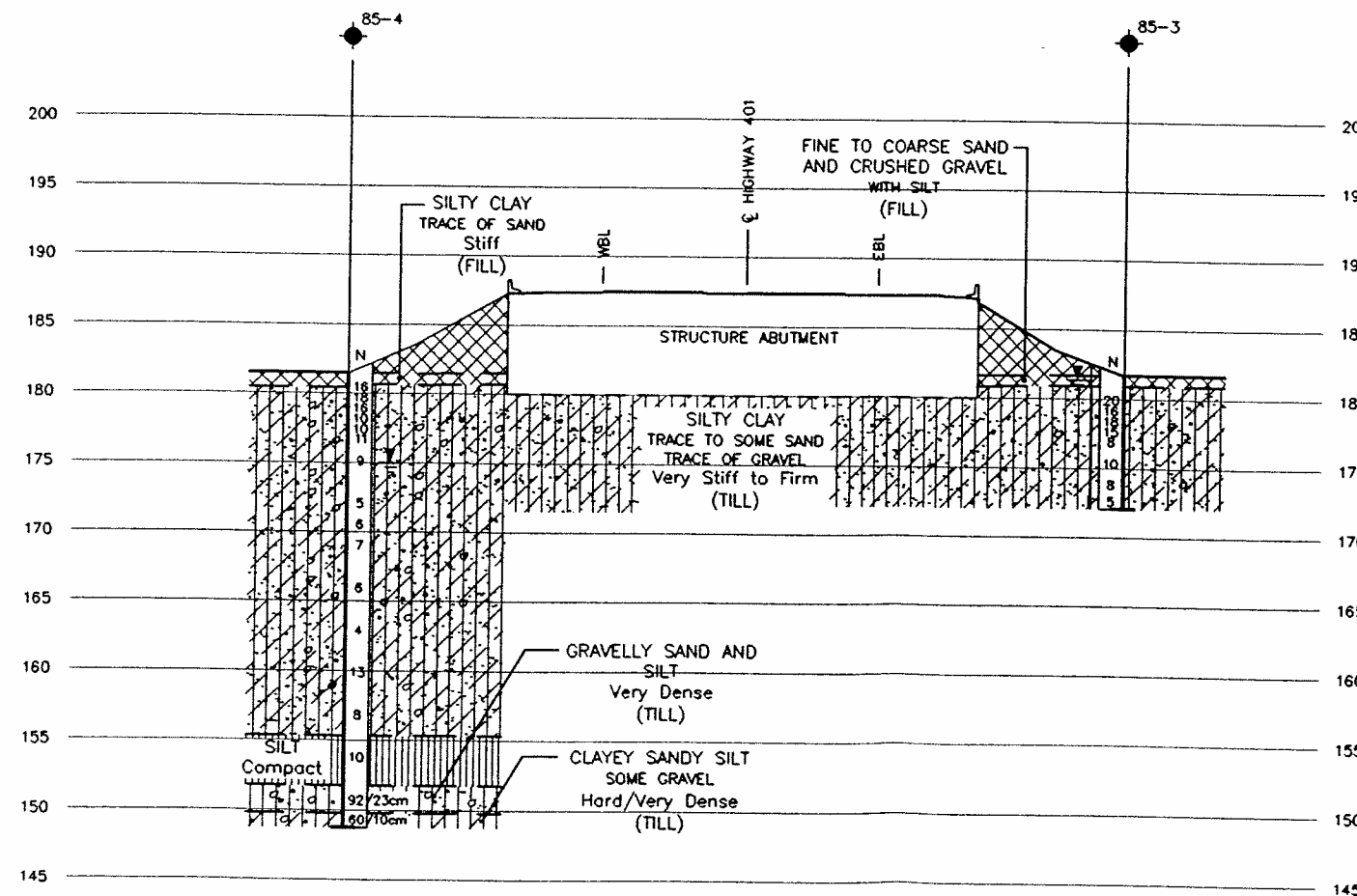
DATE	BY	DESCRIPTION

Geocres No.					
REV No	401			DRY	32
SUBM'D	01	CHECKED MRA	DATE NOV 22, 2002	SITE	6-85
DESIGN	MRA	CHECKED BRG	APPROVED DWK	DWG	1

REF No Survey Bridge Site Plan, Proposed Crossing at Belle River Road King's Highway 401, Plan Date April 2001, Survey Date January 2001, by Marshall Macklin Monaghan.



C-C



D-D

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 B-B.

SECTIONS
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 Jan - Mar 2002		
	Head		
	ARTESIAN WATER		
	Encountered		
	PIEZOMETER		
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.

DATE	BY	DESCRIPTION

Geocree No.			
REV. No	401	DATE	NOV 22, 2002
SUB. No	00	CHECKED	MRA
		DATE	NOV 22, 2002
		SITE	6-85

REF No Survey Bridge Site Plan, Proposed Crossing at Belle River Road King's Highway 401, Plan Date April 2001, Survey Date January 2001, by Marshall Macklin Monaghan

**FOUNDATION DESIGN REPORT
FOR
BELLE RIVER ROAD OVERPASS
G.W.P. 60-00-00, SITE 6-85
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: 01TF073E
Geocres No. 40J2-50

November 2002

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Deep Foundations.....	3
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APPENDICES

FIGURE 1	-	ABUTMENT ON COMPACTED FILL SHOWING GRANULAR "A" CORE
FIGURE 2	-	LATERAL EARTH PRESSURE DISTRIBUTION: SINGLY-BRACED CUTS IN COHESIVE SOILS
FIGURE 3	-	LATERAL EARTH PRESSURE DISTRIBUTION: MULTI-BRACED CUTS IN COHESIVE SOILS
FIGURE 4	-	GENERAL RECOMMENDATIONS REGARDING UNDERPINNING OF FOUNDATIONS/UTILITIES LOCATED CLOSE TO EXCAVATION

FOUNDATION DESIGN REPORT

for
Belle River Road Overpass
G.W.P. 60-00-00, Site 6-85
Highway 401
Town of Lakeshore, Ontario

INTRODUCTION

This report provides geotechnical comments and recommendations regarding design and construction of foundations and abutments for the proposed widening of the existing overpass structure at Belle River 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 existing overpass consists of a single span superstructure with a span of about 15 m and width of approximately 40 m (ref. drawing No. 401-ROC-E-01 'Proposed Crossing at Belle River Road. King's Highway 401' dated April 2001). Highway 401 will pass over Belle River Road at approximate Station 10+480, Highway 401 chainage.

Road grade on Highway 401 at the overpass location is near elevation 187.7 (interpolated from existing grade shown on the drawing referred to above) and on Belle River Road near 181.7. The approach embankments to the structure are some 3 to 5 m high and will be raised by up to 0.6 m. 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 or topsoil underlain by silty clay till overlying deposits of silt and silt/sand till. The clay till exhibits an upper stiff to hard 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.

Cognizant of the ground surface elevation and fill thickness measured at the boreholes and based on the engineering properties of the soil revealed in the boreholes drilled at the site for this study, it is assumed that the existing structure is supported on shallow spread footings founded near elevation 179.5.

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

Factored Bearing Resistance at ULS = 300 kPa

Bearing Resistance at SLS = 200 kPa

The existing embankment fill is about 5 m high. If compatible with the existing foundation design, 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/topsoil should be stripped prior to structural fill placement.

The bearing resistance for a minimum 2.5 m wide footing constructed on a minimum 3.5 m thick pad of structural fill is:

$$\begin{aligned}\text{Factored Bearing Resistance at ULS} &= 900 \text{ kPa} \\ \text{Bearing resistance at SLS} &= 350 \text{ kPa}\end{aligned}$$

The actual founding level will be subject to structural design considerations.

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:

	<u>Granular "A"</u>	<u>Granular "B"</u>
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 Earth Pressure At Rest 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 clay till overburden. The following parameters should be used in design of the system foundation:

	<u>Granular "A"</u>	<u>Granular "B"</u>	<u>Silty Clay</u>
Friction Angle, degrees	35	32	0
Cohesion, kPa	0	0	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 fill/topsoil layers. Based on the inferred design founding elevation of 179.5, the depth of excavation will vary from about 2 m beyond the toe of the embankment fill to as much as 8.0 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.

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

$$R = 0.5c_u A_s L_s$$

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

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

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

Foundations of heavily loaded/settlement sensitive structures and/or utilities located within close proximity to the excavation may require underpinning to preserve the integrity of these structures. Further comments and general recommendations in this regard are provided in Figure 4.

The groundwater level was established by means of a piezometer installed in one borehole during the field investigation. The piezometer readings showed water levels to be at a minimum depth of 0.3 m (elevation 181.4). 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.

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.



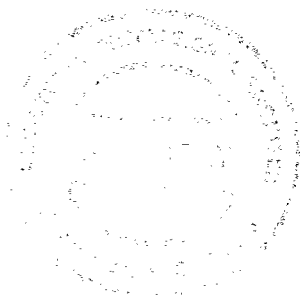
A handwritten signature in black ink, appearing to read "M. Anderson", with a long horizontal line extending to the right.

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



A handwritten signature in black ink, appearing to read "D. Kerr", with a long horizontal line extending to the right.

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



A handwritten signature in black ink, appearing to read "Brian R. Gray", with a long horizontal line extending to the right.

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

GD:lad

APPENDICES

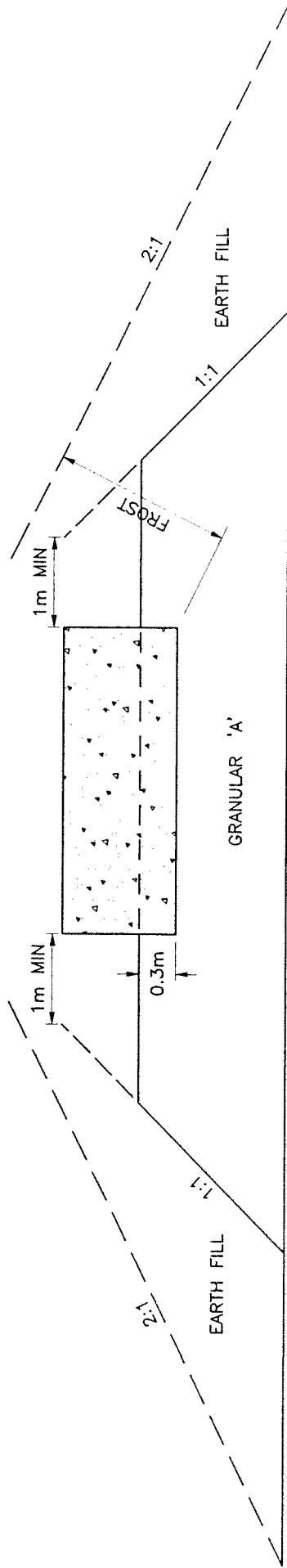
FIGURE 1 – ABUTMENT ON COMPACTED FILL SHOWING
GRANULAR "A" CORE

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

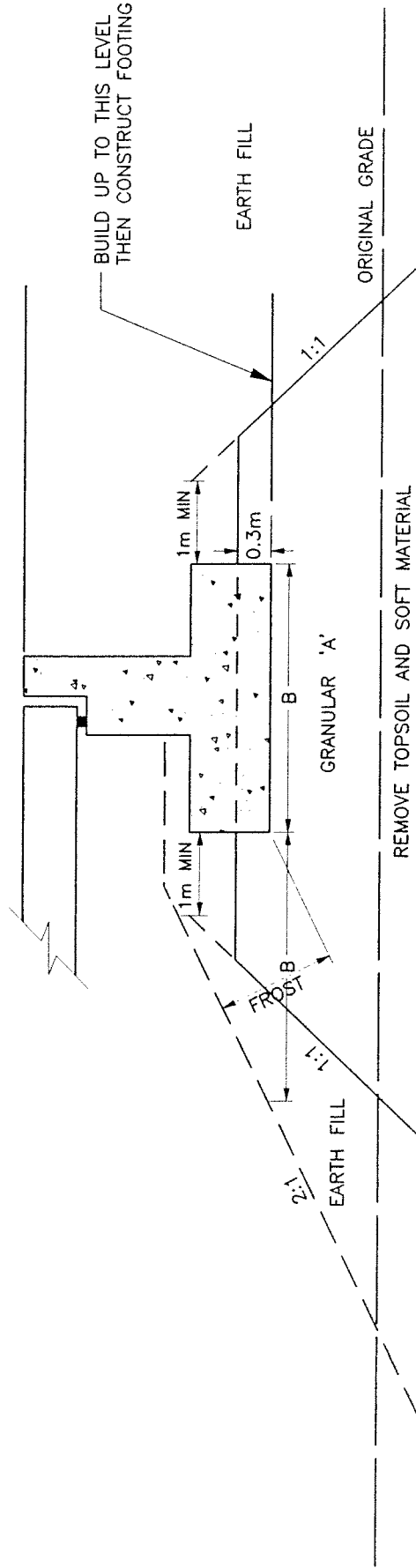
FIGURE 3 – LATERAL EARTH PRESSURE DISTRIBUTION:
MULTI-BRACED CUTS IN COHESIVE SOILS

FIGURE 4 – GENERAL RECOMMENDATIONS REGARDING
UNDERPINNING OF FOUNDATIONS/UTILITIES
LOCATED CLOSE TO EXCAVATION

ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



CROSS SECTION



LONGITUDINAL SECTION

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

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CONSULTING ENGINEERS

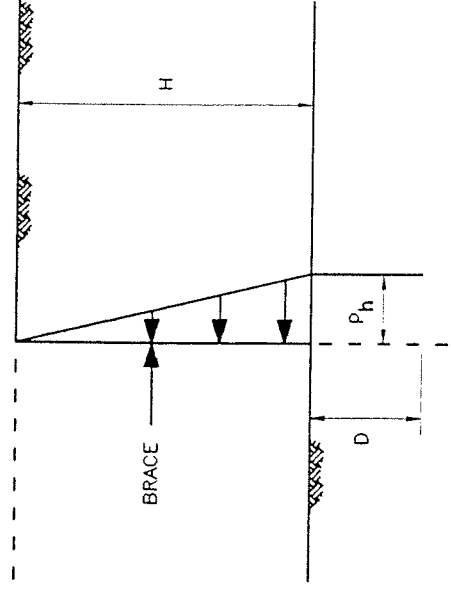
45 BURFORD ROAD, HAMILTON, ONTARIO, L8E 3C6
Tel: (905) 561-2231 Fax: (905) 561-6363

DATE	SCALE	JOB NO.	FIGURE NO.
MAR. 2002	NTS	01TF073E	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 appropriate monitoring (surface and insitu) carried out during construction.
9. Earth pressure diagram is applicable for relatively short construction periods. If excavation is to be open for long periods, monitoring of deformation is essential, the earth pressure diagram must be reviewed, and remedial works may be required.
10. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.
11. Bracing system should be regularly examined for signs of distress.
12. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.
13. This sheet should be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

EARTH PRESSURE DIAGRAM



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

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

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

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

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

RECOMMENDED DESIGN PARAMETERS

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

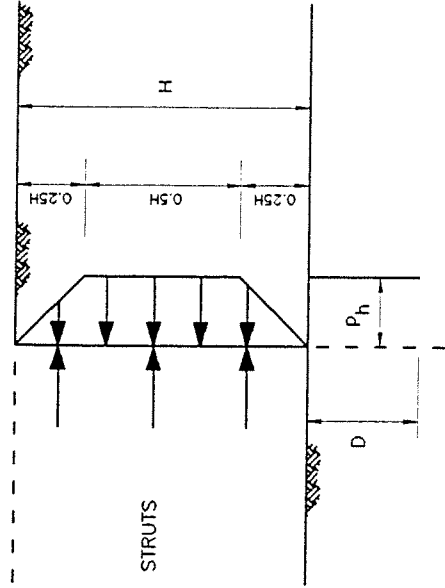
$$K = 0.35 \text{ (movement of retained soil acceptable)}$$

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

NOTES

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

EARTH PRESSURE DIAGRAM



P_h = 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

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

NOTES

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

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

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

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

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

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

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

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

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

