

**FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
WIDENING OF DUCK CREEK BRIDGE
G.W.P. 60-00-00, SITE 6-86
HIGHWAY 401
TOWN OF LAKESHORE, ONTARIO**

Distribution:

4 cc: Ministry of Transportation
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PML Ref: 01TF073F
Geocres No. 40J2-52

November 2002

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FOUNDATION INVESTIGATION REPORT

for
Widening of Duck Creek Bridge
G.W.P. 60-00-00, Site 6-86
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 Duck Creek and Highway 401 in Byrnedale, Ontario. The investigation was conducted for the Southwestern Region Structural Section of the Ontario Ministry of Transportation.

Highway 401 passes over Duck Creek at approximate Station 12+275, Highway 401 chainage, in the Town of Lakeshore (Township of Rochester). The existing bridge is a single span structure with a span of about 10 m and width of 30 m.

The report pertains to the proposed bridge structure 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 Duck Creek. The bridge structure to be widened carries Highway 401 traffic over Duck Creek. 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 30 km east of Windsor along Highway 401. The surrounding lands are mainly level and used for a mix of residential and agricultural purposes.

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

INVESTIGATION PROCEDURES

The field work was carried out during the period May 7 to 10, 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 VCP586: Top of round iron bar
 on south side of
 Highway 401 west of
 Duck Creek 18.143
 RT 12+258.275
 (Elevation 181.885 geodetic)

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

Representative samples of the soil were recovered at frequent depth intervals using a conventional split spoon sampler during drilling. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. 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 or organic layer underlain by silty clay till containing discontinuous deposits of silt and sand. The strata encountered are summarized below.

Fill

Surficial fill was present in boreholes 86-2 to 86-4. The fill was 1.4 m thick and consisted of stiff silty clay, its moisture content being 24 and 27%. Inclusions of topsoil were identified within the fill in borehole 86-3.

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

Topsoil/Alluvium

Topsoil/alluvium was present surficially in borehole 86-1. This unit was 1.1 m thick and consisted of silty clay.

Silty Clay

Directly beneath the fill at elevation 180.2 in borehole 86-3 was cohesive silty clay. This deposit was 800 mm thick and stiff in consistency. The moisture content of the deposit was 23%.

Silty Clay Till

Cohesive silty clay till was encountered in all the boreholes at depths of 1.1 to 2.2 m (elevation 178.7 to 180.2). The consistency of the clay till was stiff to hard, typically stiff to very stiff. Standard penetration test 'N' values ranged from 10 to 43, generally being in a range of 10 to 20. The results of pocket penetrometer testing carried out in this stratum at various depths indicate that the undrained shear strength varies between 50 and 200 kPa.

The moisture content of the clay till typically ranged from 17 to 21%, increasing locally to 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 34 and 47 and plastic limits of 15 and 21. The results of particle size distribution analyses conducted on the clay till are presented in Figure 2 (Appendix A).

The clay till had a thickness of 7.2 m in borehole 86-2. However, this stratum is expected to continue further down, since it was not penetrated upon termination of the remaining boreholes at 9.6 m depth (elevation 170.2 to 172.0).

Clayey Silt Till

Two layers of cohesive clayey silt till were revealed within the silty clay till in borehole 86-1. The first layer was 700 mm thick and encountered at 5.6 m depth (elevation 174.2), the second layer was 800 mm thick and encountered at 7.0 m depth (elevation 172.8). The consistency of both layers was very stiff. In borehole 86-2, clayey silt till revealed at 8.6 m depth (elevation 172.6) was hard and had a moisture content of 13%. Borehole 86-2 was terminated in this deposit at 9.6 m depth (elevation 171.6).

Silt and Sand

In borehole 86-1, a 700 mm thick layer of silt and fine sand was revealed within the clayey silt till at 6.3 m depth (elevation 173.5). The silt and sand was cohesionless and very dense.

Groundwater

Groundwater was measured in boreholes 86-1 and 86-2 at respective depths of 1.5 and 7.4 m (elevation 178.3 and 173.8) at the completion of drilling. Water was not observed in the remaining boreholes.

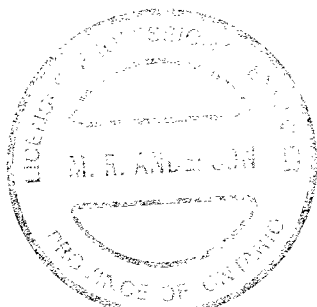
Based on these measured water levels along with visual examination of the samples retrieved during drilling and water level observations/measurements during field investigations conducted for other structures throughout the study corridor, the stabilized groundwater level at this site is expected to be near the water level in the adjacent creek, elevation 179.2 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.

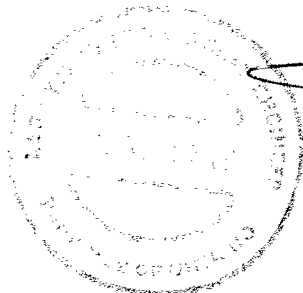
A handwritten signature of Murray R. Anderson in black ink.

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.

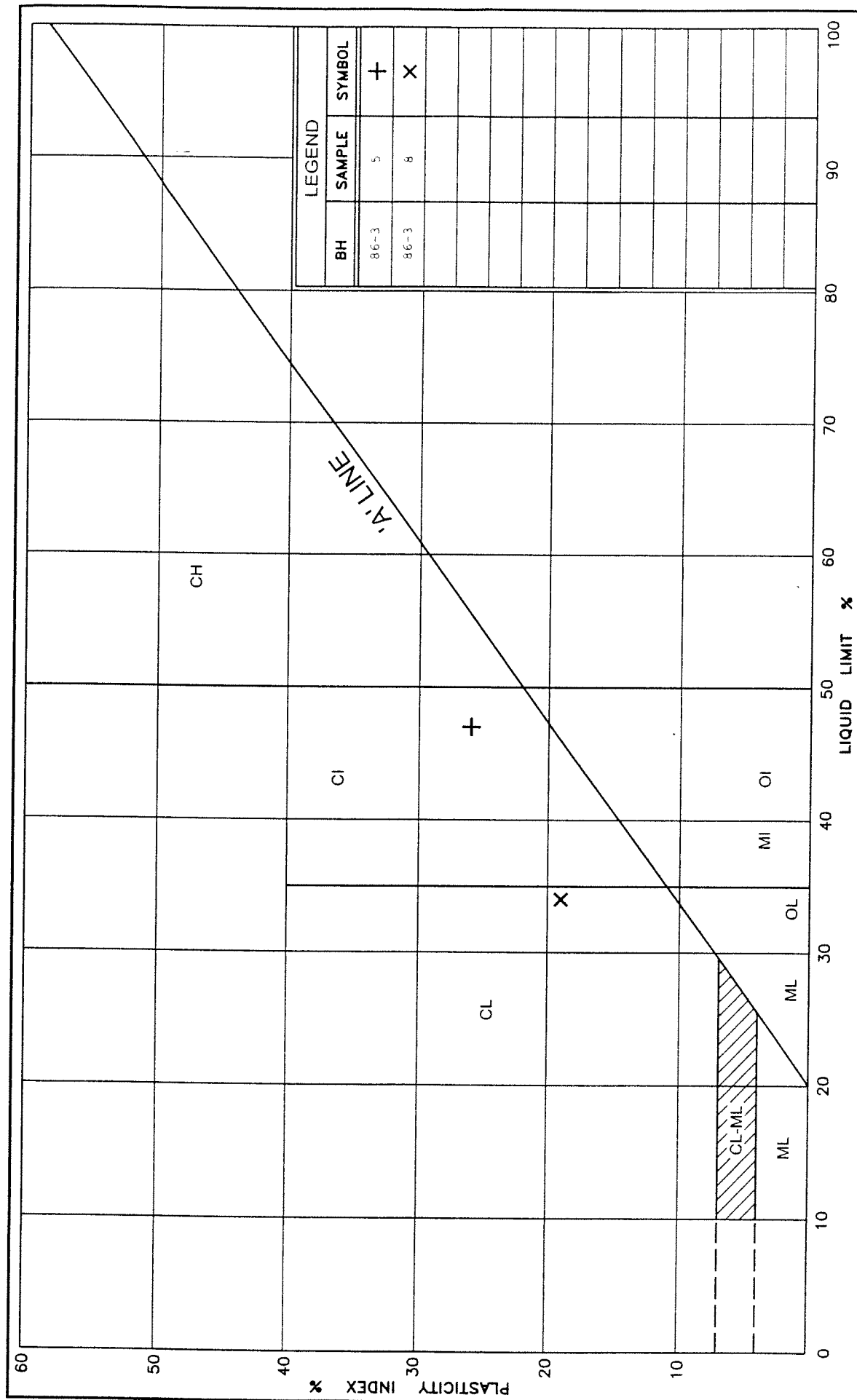
Brian R. Gray, M.Eng., P.Eng.
President

GD:lad

APPENDIX A

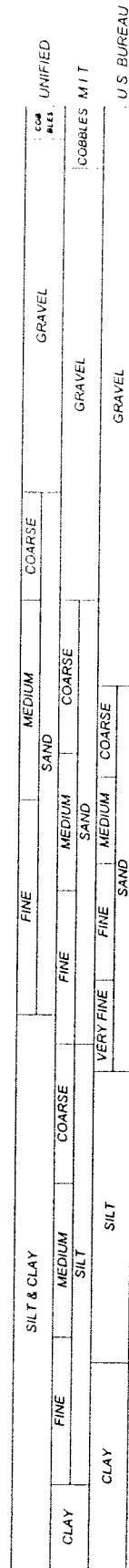
Figure 1 – Plasticity Chart

Figure 2 – Grain Size Distribution Chart



	PLASTICITY CHART		FIG No 1
	SILTY CLAY, some sand, trace of gravel (CI)		HIGHWAY 401
	G.W.P. 60-00-00, Site 6-86		

U.S. STANDARD SIEVE SIZES

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

APPENDIX B

Record of Borehole Sheets

Drawing 1

LIST OF ABBREVIATIONS

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

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

TYPE OF SAMPLE

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

SOIL TESTS

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

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

■ - UNDRAINED SHEAR STRENGTH DETERMINED FROM POCKET PENETROMETER TEST.

RECORD OF BOREHOLE No 86-1

1 of 1 METRIC

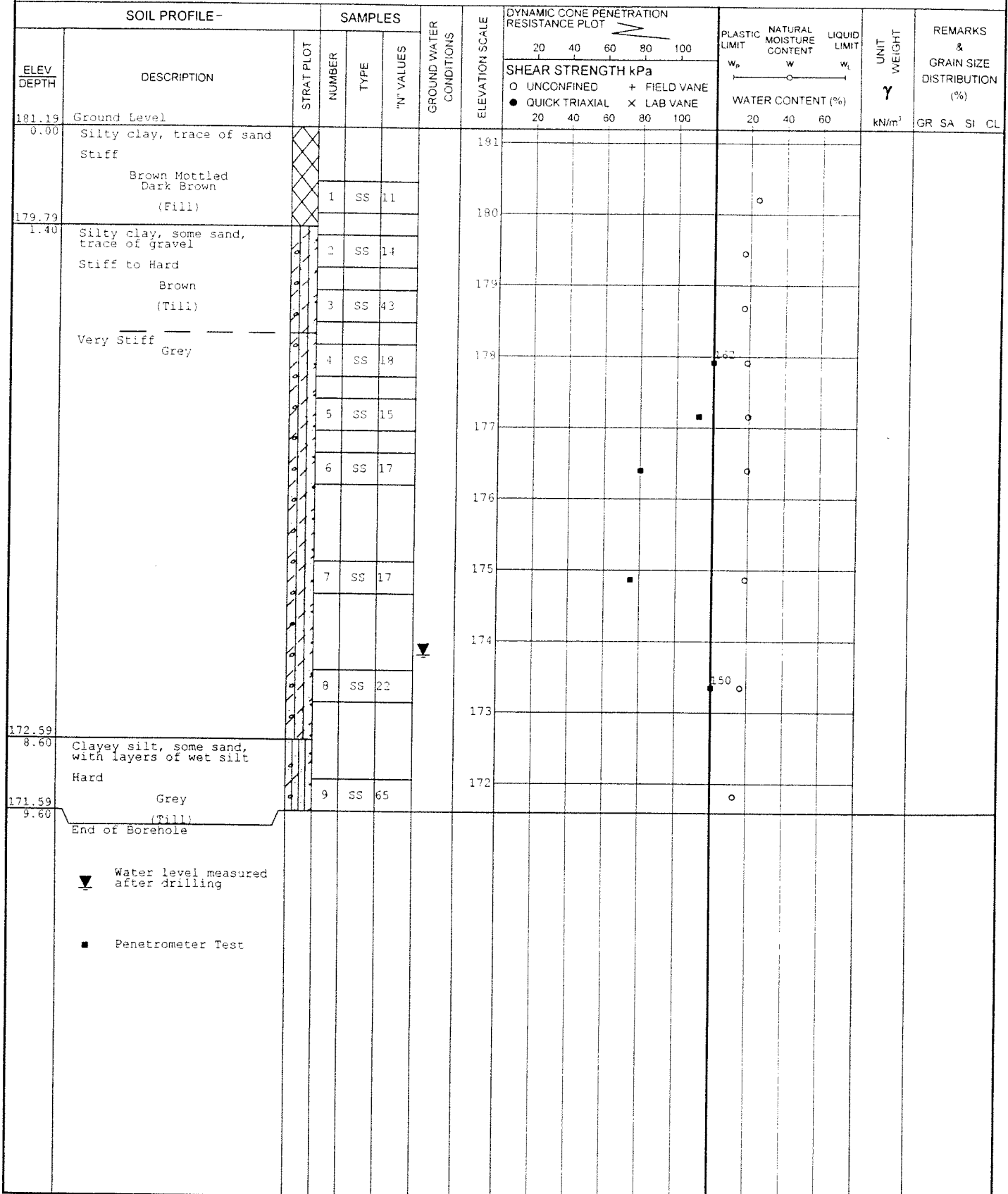
G.W.P. 60-00-00 LOCATION Co-ords. 1 677 440 N; 184 044 E. ORIGINATED BY MR
DIST 10 HWY 401 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY JD
DATUM Geodetic DATE May 10, 2002 CHECKED BY MPA

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						
179.76	Ground Level							○ UNCONFINED	+ FIELD VANE					
0.00	Silty clay							● QUICK TRIAXIAL	x LAB VANE					
	Dark Brown Topsoil/Alluvium													
178.66			1	SS	5		179							
1.10	Silty clay, some sand, trace of gravel													
	Stiff to Very Stiff		2	SS	22		178							
	Brown (Till)													
	Grey		3	SS	18		177							
			4	SS	15									
			5	SS	19		176							
			6	SS	18		175							
174.16														
5.60	Clayey silt, some sand						174							
	Very Stiff													
173.46	Grey		7	SS	82		173							
6.30	(Till)													
172.76	Silt and Fine Sand													
7.00	Very Dense													
	Grey													
	Clayey silt, some sand													
171.96	Very Stiff		8	SS	25		172							
7.80	Grey													
	(Till)													
	Silty clay, some sand, trace of gravel													
	Very Stiff to Stiff						171							
	Grey		9	SS	13									
	(Till)													
170.16														
9.60	End of Borehole													
	Water level measured after drilling													
	Penetrometer Test													

RECORD OF BOREHOLE No 86-2

1 of 1 METRIC

G.W.P. 60-00-00 LOCATION Co-ords. 4 672 404 N; 289 035 E. ORIGINATED BY MR
DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY JCD
DATUM Geodetic DATE May 07, 2002 CHECKED BY MRA



RECORD OF BOREHOLE No 86-3

1 of 1 METRIC

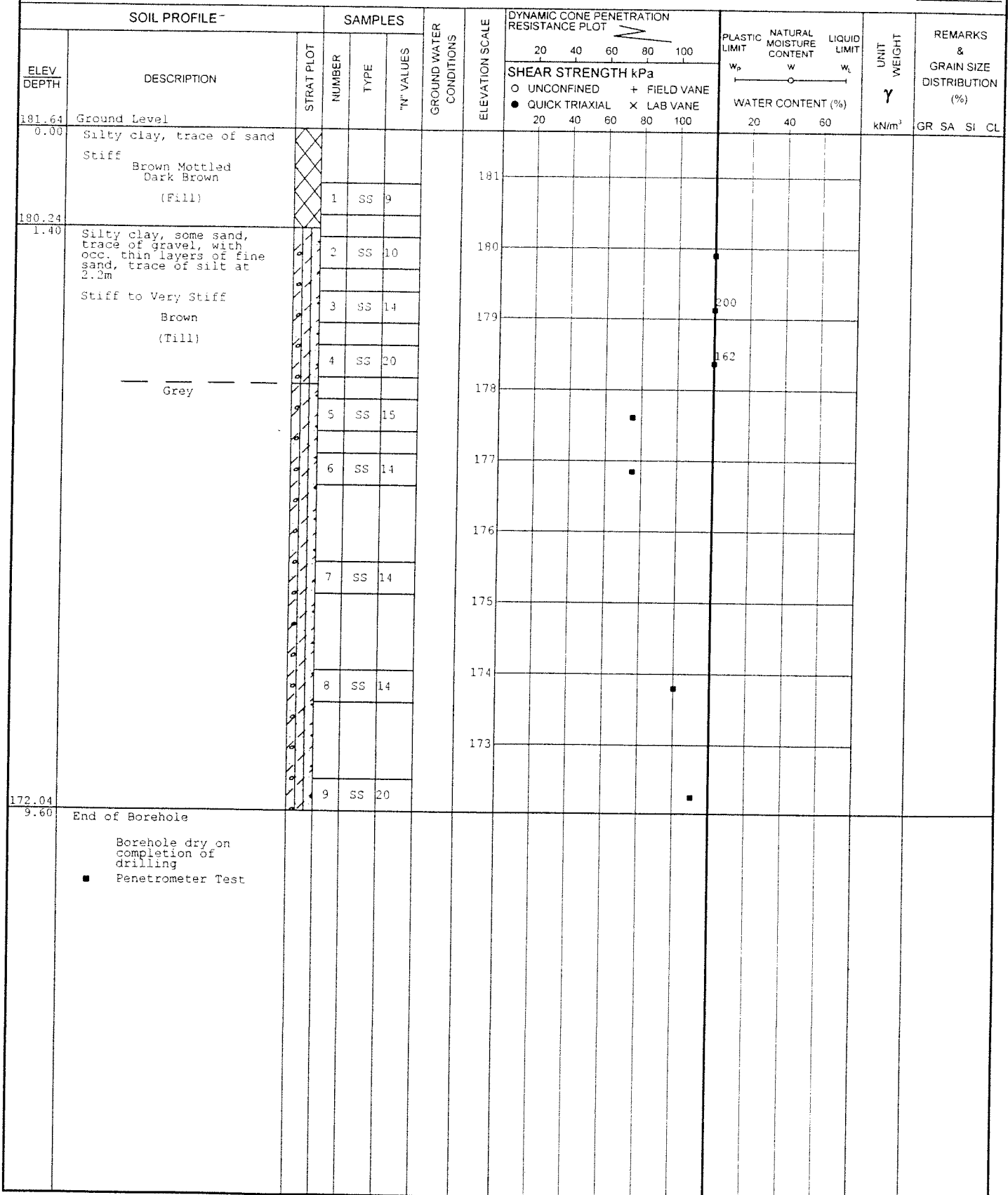
G.W.P. 60-00-00 LOCATION Co-ords. 1 677 449 N; 299 057 E. ORIGINATED BY MR
DIST 32 HWY 401 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY GD
DATUM Geodetic DATE May 10, 2002 CHECKED BY MRA

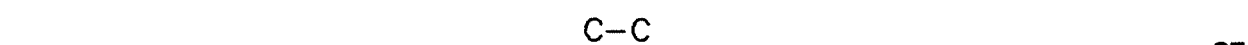
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
181.62 0.00	Ground Level Silty clay/Topsoil Stiff Brown to Dark Brown (Fill)		1	SS	12		181							
180.22 1.40	Silty clay, trace of sand Stiff Brown		2	SS	9		180							
179.42 2.20	Silty clay, some sand, trace of gravel Stiff to Very Stiff Brown (Till)		3	SS	13		179							
			4	SS	16		178							
			5	SS	29		177							1 15 46 38
	Grey		6	SS	15		176							
			7	SS	18		175							
			8	SS	12		174							1 16 46 37
			9	SS	11		173							
172.02 9.60	End of Borehole Borehole dry on completion of drilling ■ Penetrometer Test													

RECORD OF BOREHOLE No 86-4

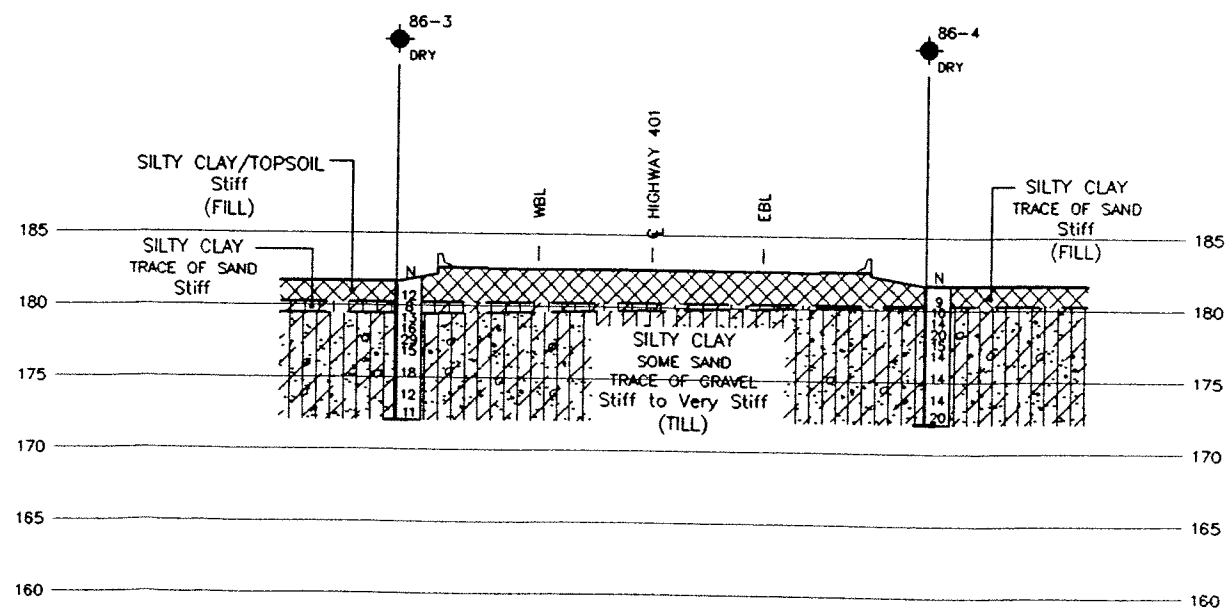
1 of 1 METRIC

G.W.P. 60-00-00 LOCATION Co-ords. 4 677 403 N; 289 050 E. ORIGINATED BY MR
DIST 32 HWY 101 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY GD
DATUM Geodetic DATE May 07, 2002 CHECKED BY MRA





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.



SECTIONS
SCALE



D-D

REF No Survey Plan 2001 Site # 6-86, entitled Proposed Crossing at Duck Creek and King's Highway 401, Prepared by Marshall Macklin Monaghan and Planning and Design Section, MTO.

METRIC

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

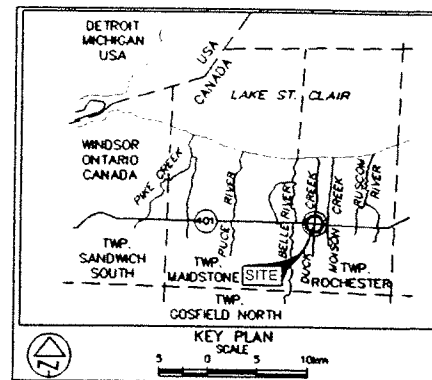
CONT No

GWP No 60-00-00








HIGHWAY 401

PROPOSED BRIDGE WIDENING AT DUCK CREEK
BOREHOLE LOCATIONS & SOIL STRATA

Peto MacCallum Ltd
CONSULTING ENGINEERS



LEGEND

	Borehole
	Dynamic Cone Penetration Test (Cone)
	Borehole & Cone
N	Blows/0.3m (Std. Pen Test, 475 J / blow)
COONE	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
86-1	179.76	4 677 442	289 044
86-2	181.19	4 677 404	289 035
86-3	181.62	4 677 440	289 055
86-4	181.64	4 677 403	289 050

- NOTE -

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

[illegible]

Census No.			
ENTRY No	401	DIST	32
SUB/D	00	CHECKED MRA	DATE NOV 21, 2002
FILE NO	00000000	RECEIVED BY:	APPROVED FILE#
			6-88
			1

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FOR
WIDENING OF DUCK CREEK BRIDGE
G.W.P. 60-00-00, SITE 6-86
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APPENDICES

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

FOUNDATION DESIGN REPORT

for
Widening of Duck Creek Bridge
G.W.P. 60-00-00, Site 6-86
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 Duck Creek and Highway 401 in Byrnedale, 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-ROC-E-02 'Proposed Crossing at Duck Creek, King's Highway 401' dated April 2001). Highway 401 will pass over Duck Creek at approximate Station 12+275, Highway 401 chainage, in the Town of Lakeshore (Township of Rochester).

Road grade on Highway 401 at the bridge location is near elevation 182.7 (interpolated from existing grade shown on the drawing referred to above). The existing approach embankments comprise fill with a height of approximately 2.0 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 generally comprised a surficial fill or organic layer underlain by silty clay till containing discontinuous deposits of silt and sand.

FOUNDATIONS

Spread Footings

Supporting the structure widenings on conventional spread footings founded in the native mineral soil is considered to be feasible at this site. The new footings should be founded in the stiff to very stiff clay till deposit at the same level as the existing footings.

Details concerning the design of the foundations supporting the existing structure were not provided. 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 178.5.

The bearing resistance of 2.5 m wide footings constructed on the very stiff to hard clay till at the inferred design founding level is as follows:

Factored Bearing Resistance at ULS = 300 kPa

Bearing Resistance at SLS = 200 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 2 m thick and a minimum 3.5 m thickness of engineered fill would be required to optimize the bearing resistance values at ULS and SLS.

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

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

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

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

Deep Foundations

Supporting the structure widenings on deep foundations is not recommended considering the potential for incompatible settlement behaviour between the new and existing portions of the 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 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 clay till. The following parameters should be used in design of the system foundation:

	<u>Granular "A"</u>	<u>Granular "B"</u>	<u>Silty Clay</u>
Friction Angle, degrees	35	32	0
Cohesion, kPa	0	0	100
Unit weight, kN/m ³	22.8	21.2	20.4

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

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

APPROACH EMBANKMENTS

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

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

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

EXCAVATION AND GROUNDWATER CONTROL

Excavation for construction of footings is expected to extend through the fill, topsoil and alluvium layers. Based on the inferred design founding elevation of 178.5, the depth of excavation will vary from about 3 m beyond the toe of the embankment fill to as much as 4.5 m at the top of the fill. Cognizant of the fill present at the site, the materials are classified as Type 3 soils according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Temporary cut slopes inclined at 45° to the horizontal should generally be stable. Flatter side slopes may be required if excessively soft/wet materials or concentrated seepage zones are encountered locally.

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

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

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:

$$R = 0.45c_u A_s L_s$$

where

c_u = average undrained shear strength over the anchor length

= 100 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:

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

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

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

The stabilized groundwater level at this site is expected to be near the water level in the creek – elevation 179.2 when the field investigation was conducted. Considering the low permeability characteristics of the soil, groundwater seepage or surface water that enters the excavation should be readily handled by conventional sump pumping techniques.

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

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.



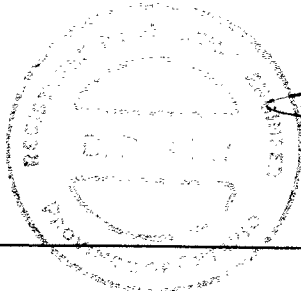
Yours very truly

Peto MacCallum Ltd.

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

APPENDICES

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

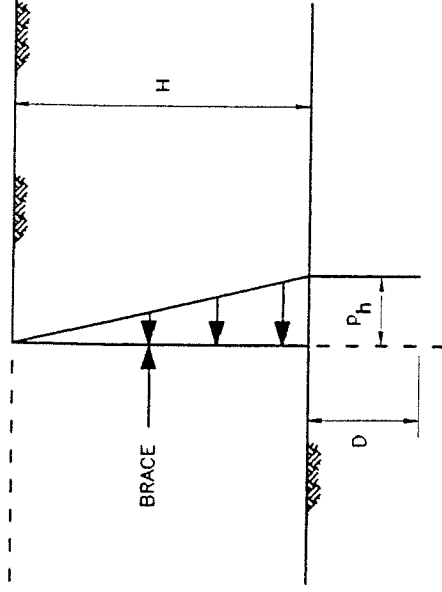
FIGURE 2 – LATERAL EARTH PRESSURE DISTRIBUTION:
MULTI-BRACED CUTS IN COHESIVE SOILS

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

NOTES

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

EARTH PRESSURE DIAGRAM



- P_h = 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).

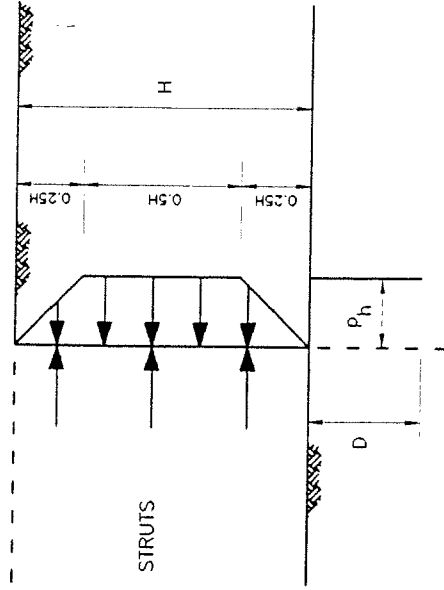
RECOMMENDED DESIGN PARAMETERS

- $\gamma = 20.4 \text{ kN/m}^3$
 $K = 0.35$ (movement of retained soil acceptable)
 0.50 (movement of adjacent structures/facilities unacceptable)

NOTES

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



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

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.

