

Golder Associates Ltd.

309 Exeter Road, Unit 1
London, Ontario, Canada N6L 1C1
Telephone 519-652-0099
Fax 519-652-6299



**FOUNDATION INVESTIGATION AND DESIGN REPORT
PUCE RIVER BRIDGE REPLACEMENT
HIGHWAY 401, SITE 6-083
FROM 1.5 KILOMETRES WEST OF MANNING ROAD
EASTERLY TO 1.3 KILOMETRES EAST OF PUCE ROAD
GWP 62-00-00, AGREEMENT NO. 3005-A-000393
MINISTRY OF TRANSPORTATION - SOUTHWESTERN REGION**

Submitted to:

Dillon Consulting Limited
P.O. Box 426, Terminal B
495 Richmond Street
London, Ontario
N6A 4W7

DISTRIBUTION:

- 8 Copies - Dillon Consulting Limited
- 2 Copies - Golder Associates Ltd.

December 24, 2004

041-130054-0-3
Geocres No. 40J2-65



TABLE OF CONTENTS

<u>SECTION</u>	<u>PAGE</u>
PART A – FOUNDATION INVESTIGATION REPORT	
1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	1
2.1 General	1
2.2 Site Geology	1
3.0 INVESTIGATION PROCEDURES	2
4.0 SUBSURFACE CONDITIONS	3
4.1 Site Stratigraphy	3
4.1.1 Topsoil and Fill Materials	4
4.1.2 Sandy Silt	4
4.1.3 Silty Clay Till	5
4.1.4 Silty Clay	6
4.1.5 Sand	6
4.1.6 Silty Sand	6
4.1.7 Silt	6
4.1.8 Clayey Silt Till	7
4.1.9 Bedrock	7
4.2 Groundwater Conditions	7
5.0 MISCELLANEOUS	8
PART B – FOUNDATION DESIGN REPORT	
6.0 ENGINEERING RECOMMENDATIONS	9
6.1 General	9
6.2 Bridge Foundations	9
6.3 Shallow Foundations	10
6.3.1 Axial Geotechnical Resistance	10
6.3.2 Resistance to Lateral Forces	10
6.3.3 Frost Protection	10
6.3.4 Construction Considerations	11
6.4 Deep Foundations	11
6.4.1 Geotechnical Axial Resistance	11
6.4.2 Downdrag Load (Negative Skin Friction)	11
6.4.3 Resistance to Lateral Loads	12
6.4.4 Frost Protection	13
6.5 Lateral Earth Pressures	13
6.6 Embankments	14
6.7 Excavations and Temporary Cut Slopes	14

TABLE OF CONTENTS CONTINUED

LIST OF ABBREVIATIONS

LIST OF SYMBOLS

RECORDS OF BOREHOLES

FIGURE 1 – Key Plan

DRAWING 1 – Borehole Locations

DRAWING 2 – Soil Strata

DRAWING 3 – Soil Strata

APPENDIX A – Laboratory Test Data (Figures A-1 to A-4)

APPENDIX B – Records of Boreholes by Others

APPENDIX C – Site Photographs

PART A

**FOUNDATION INVESTIGATION REPORT
PUCE RIVER BRIDGE REPLACEMENT
HIGHWAY 401, SITE 6-083
FROM 1.5 KM WEST OF MANNING ROAD
EASTERLY TO 1.3 KM EAST OF PUCE ROAD
GWP 62-00-00, AGREEMENT NO. 3005-A-000393
MINISTRY OF TRANSPORTATION – SOUTHWESTERN REGION**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Dillon Consulting Limited on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services as part of the detail design work for the section of Highway 401 described by GWP 62-00-00. This section of Highway 401 is some 9.9 kilometres in length and extends from 1.5 kilometres west of Manning Road in the Township of Sandwich South to 1.3 kilometres east of Puce Road in the Township of Maidstone, Ontario.

The purpose of this portion of the foundation investigation was to determine the subsurface conditions for the replacement of the Puce River bridge (MTO Site 6-083) by drilling boreholes and carrying out in-situ and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's request for proposal and in Golder Associates proposal P31-3115-2, dated February 2, 2004. The work was carried out in accordance with our Quality Control Plan for Foundation Engineering Detail Design Services dated March 29, 2004.

2.0 SITE DESCRIPTION

2.1 General

GWP 62-00-00 comprises the reconstruction and widening of some 9.9 kilometres of Highway 401 extending from 1.5 kilometres west of Manning Road easterly to 1.3 kilometres east of Puce Road. The location of the project is shown on the Key Plan, Figure 1. The project chainage extends from Highway 401 Station 18+675 (Township of Sandwich South) to Station 18+369 (Township of Maidstone). It should be noted that the Township boundary lies along the centerline of Manning Road and the Highway 401 chainage equation at this location is Station 20+174.709 (Township of Sandwich South) equals Station 10+000 (Township of Maidstone).

This report addresses the subsurface conditions for the reconstruction of the Puce River bridge. The location of the subject bridge is shown on the Key Plan, Figure 1.

2.2 Site Geology

The project lies within the Essex Clay Plain, a subregion of the physiographic region of southern Ontario known as the St. Clair Clay Plains, identified in "The Physiography of Southern Ontario" by Chapman and Putnam (1984). The clay plain is described as a till plain that has been smoothed by shallow deposits of lacustrine clay which settled in the depressions of the till. The prevailing soil type is reportedly the Brookston clay.

Based on the Ontario Department of Mines and Northern Affairs Preliminary Maps P.749 and P.750 entitled "Quaternary Geology of the Windsor-Essex Area" Western and Eastern Parts,

respectively, the project area is reportedly located in predominantly clayey silt till. At the Manning Road interchange, a thin and discontinuous glaciolacustrine medium sand layer reportedly overlies the clayey silt till in the southeast, southwest and part of the northwest quadrants.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between July 23 and August 24, 2004, during which time two boreholes were drilled at the locations indicated on the Location Plan, Drawing 1.

The as-drilled borehole locations, ground surface elevations and borehole depths are as follows:

<u>BOREHOLE</u>	<u>LOCATIONS (m)</u>		<u>GROUND SURFACE</u>	<u>BOREHOLE DEPTH</u>
	<u>Northing</u>	<u>Easting</u>	<u>ELEVATION</u> (m)	
A	4677674.7	280471.8	184.51	14.17
B	4677650.6	280470.8	184.06	34.99

The soil stratigraphy encountered in the boreholes is shown on the attached Record of Borehole sheets.

The boreholes were advanced using all terrain vehicle mounted power augers owned and operated by a specialist drilling contractor. Samples of the overburden were obtained at suitable intervals of depth using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures, or a 80 millimetre outside diameter thin walled open Shelby tube sampler. In situ vane testing was carried out, where possible, within the cohesive strata. Groundwater conditions were observed in the open boreholes throughout the drilling operations. All of the boreholes were backfilled in accordance with current regulations and MTO recommended procedures.

The field work was supervised on a full-time basis by experienced members of our engineering staff who arranged for utility locates, directed the drilling, sampling and in situ testing operations, logged the boreholes and cared for the samples obtained. The soil samples were identified in the field, placed in labeled containers and transported to Golder Associates' London laboratory for further examination and routine testing. Routine index and classification tests consisting of water content determinations, grain size distribution analyses and Atterberg limits determinations were carried out on selected samples. Selected Shelby Tube samples were forwarded to Golder Associates' Mississauga laboratory for consolidation testing. The results of the field and laboratory testing are given on the Record of Borehole sheets and in Appendix A.

Temporary traffic control was carried out in accordance with the Ontario Traffic Manual, Temporary Conditions, Book 7, dated March 2001.

The as-drilled borehole locations and ground surface elevations were surveyed by members of our engineering staff. The elevations at the borehole locations were referenced to a benchmark provided by Dillon Consulting Limited. The locations of the boreholes are indicated on the Record of Borehole sheets and on Drawing 1, attached.

The results of boreholes from previous investigations carried out by Peto MacCallum Ltd. and the MTO at the Puce Creek bridge have been included in this report. The boreholes are:

- Boreholes 83-1 and 83-2, Geocres No. 40J2-54, Report No. 01TH073C entitled "Foundation Investigation Report for Widening of Puce River Bridge, G.W.P. 60-00-00, Site 6-83, Highway 401, Town of Lakeshore, Ontario", dated November 2002, prepared by Peto MacCallum Ltd.
- Boreholes 1 and 2 which were drilled on July 15 and 16, 1965 by the MTO for a previous widening on the south side of the structure (W.P. 693-64). Logs for these boreholes were reproduced in the above-noted Peto MacCallum Ltd. Report.

Boreholes, MTO 1 and MTO 2 were advanced at the south side of the bridge to depths of 13.1 and 6.6 metres, respectively. Dynamic cone penetration testing was conducted in the upper portion of borehole MTO 1. The borehole locations are shown on the Department of Highways - Ontario Drawing No. TWP 108-83-2-B dated August 18, 1965. Peto MacCallum boreholes, 83-1 and 83-2 were drilled on May 13, 2002 at the north side of the existing bridge to depths of 11.1 and 9.6 metres.

The approximate borehole locations are shown in plan on Drawing 1. The subsurface conditions encountered in the boreholes drilled along the north and south abutments are shown on Drawings 2 and 3 in metric units. The results of the boreholes drilled for the existing structure are provided in Appendix B in their original units and format. Corresponding depths and elevations in metric units have been added to the Records of Boreholes from the 1965 investigation by others.

4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of the in situ and laboratory testing are given on the attached Record of Borehole sheets following the text of this report, and in Appendices A and B. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and represent transitions between soil types rather than exact planes of geological change. Subsurface conditions may vary significantly between and beyond the borehole locations.

In summary, the boreholes drilled for the Puce River bridge replacement encountered topsoil and fill materials underlain by layers of sandy silt, silty clay till, silty clay, silt, silty sand, sand and clayey silt till overlying bedrock.

A detailed description of the subsurface conditions encountered in the boreholes is provided on the Record of Borehole sheets and is summarized in the following sections.

4.1.1 Topsoil and Fill Materials

Topsoil was encountered at ground surface in boreholes A and B. The topsoil was some 90 and 210 millimetres thick.

Beneath the topsoil in borehole A, a layer of granular base materials some 430 millimetres thick was encountered. The granular base materials contained pieces of asphalt.

Beneath the topsoil in borehole B, the granular base material in borehole A, and at the ground surface in borehole 83-1 earth fill materials were encountered. The fill consisted of sandy silt in borehole A, silty clay in borehole B and the upper 1.4 metres of borehole 83-1 and sandy clayey silt under the silty clay fill layer in borehole 83-1. The fill ranged in thickness from 1.9 to 2.2 metres and had measured N values, as determined in the standard penetration testing, between 9 and 21 blows per 0.3 metres of penetration. The fill materials had water contents of about 17 per cent.

4.1.2 Sandy Silt

Below the fill in borehole A, the silty clay till in borehole B and within the clayey silt till layer in borehole MTO 1, layers of sandy silt some 0.9 to 1.8 metres thick, were encountered from elevations 172.1 and 182.4 metres.

The sandy silt was loose with measured N values between 5 and 10 blows per 0.3 metres. Blow counts of approximately 85 to 115 blows per 0.3 metres were measured within the clayey silt till during dynamic cone penetration testing conducted adjacent to borehole MTO 1. The sandy silt had water contents ranging from 12 to 23 per cent with an average water content of about 17 per cent.

A grain size distribution curve for a sample of the sandy silt recovered from borehole B is provided on Figure A-1.

4.1.3 Silty Clay Till

Beneath the silty sand in borehole A, the fill in boreholes B and 83-1 and at the ground surface in boreholes 83-2, MTO 1 and MTO 2, layers of silty clay till were encountered. Borehole A was terminated in a layer of silty clay till after exploring it for some 1.4 metres below elevation 171.7 metres. Where fully penetrated, the silty clay layers were approximately 4.6 to 9.8 metres thick and were intercepted from elevations 181.5 metres to 183.7 metres. Grain size distribution curves for samples of the silty clay till recovered from the standard penetration testing are shown on Figure A-2.

In boreholes A, B and MTO 2, the upper crust of the very stiff silty clay till was some 1.2 to 3.7 metres thick. The upper crust had measured N values between 12 and 29 blows per 0.3 metres with an average N value of about 18 blows per 0.3 metres.

The lower layer of firm to very stiff silty clay till encountered in boreholes A, B and MTO 2 and was 6.9 to 7.8 metres thick. The lower layer of the silty clay till had N values between the weight of the sampling hammer and 12 blows per 0.3 metres. In the remaining boreholes, N values ranged between 10 to 89 blows per 0.3 metres. In situ vane testing carried out in the softer portions of the silty clay till at and below elevation 170 metres indicated undrained shear strengths ranging from 80 to greater than 144 kilopascals. Shear strengths of 50 to 75 kilopascals were estimated from unconfined compressive strength testing conducted on Shelby tube samples retrieved from boreholes MTO 1 and MTO 2. Blow counts of approximately 13 to 50 blows per 0.3 metres were measured within the silty clay till during dynamic cone penetration testing conducted adjacent to borehole MTO 1.

The natural water contents of the silty clay till ranged from 14 to 32 per cent with an average of 20 per cent. The silty clay till had an average plastic and liquid limits of 16 and 35 per cent, respectively, based on the Atterberg limit determinations carried out on two samples. The results are plotted on the plasticity chart, Figure A-3, which shows the deposit to be an inorganic clay of low to intermediate plasticity. Previous testing indicated a low to intermediate plasticity clay with plastic and liquid limits of 16 to 22 per cent and 28 to 42 per cent, respectively.

The results of a laboratory consolidation test carried out on sample 9 from borehole B are provided on Figures A-4 and A-5 in Appendix A. The results indicate that the silty clay till is slightly preconsolidated by about 60 kilopascals beyond the existing overburden pressure. The following table summarizes the relevant oedometer test results:

BOREHOLE AND SAMPLE	DEPTH (m)	σ'_{po} (kPa)	σ'_p (kPa)	OCR	e_0	C_r	C_c	C_v cm ² /sec
B-9	7.9	135	200	1.48	0.59	0.044	0.150	2x10 ⁻³

4.1.4 Silty Clay

Within the silty clay till layer in borehole A, beneath the sandy silt in borehole B and beneath the silty clay till in borehole 83-1, layers of firm to very stiff silty clay were encountered. Borehole 83-1 was terminated in a layer of silty clay with numerous silt lenses and partings after exploring some 0.9 metres from elevation 174.4 metres. Where fully penetrated, the silty clay layer was between 0.8 and 15.5 metres thick and intercepted from elevations 170.3 and 172.5 metres.

The silty clay had measured N values between the weight of the sampling hammer and 8 blows per 0.3 metres in boreholes A and B and 25 blows per 0.3 metres in borehole 83-1. The natural water content of the silty clay ranged from 23 to 35 per cent. In situ vane testing carried out in the silty clay indicated undrained shear strengths ranging from 46 to 63 kilopascals and a sensitivities between 1.5 and 2.4.

The silty clay had plastic and liquid limits of 25 and 42 per cent, respectively. These results are plotted on the plasticity chart, Figure A-3 which shows the deposit to be an inorganic clay of intermediate plasticity.

4.1.5 Sand

Beneath the silty clay in borehole B, a layer of dense sand some 2.6 metres thick was encountered from elevation 154.8 metres. The sand had a measured N value of 36 blows per 0.3 metres and a natural water content of about 18 per cent.

4.1.6 Silty Sand

Borehole MTO 2 was terminated in a layer of dense silty sand after exploring some 1.4 metres below elevation 178.0 metres. An approximately 150 millimetre layer or pocket of dense silty sand was encountered from elevation 171.2 metres in borehole MTO 1. The silty sand had a measured N value of 37 blows per 0.3 metres.

4.1.7 Silt

Borehole 83-2 was terminated in a layer of compact silt after exploring some 2.5 metres below elevation 175.60 metres. The silt had measured N values of 11 and 30 blows per 0.3 metres and natural water contents of about 22 per cent.

4.1.8 Clayey Silt Till

Beneath the sand in borehole B and the silty clay till in borehole MTO 1, layers of stiff to hard clayey silt till were encountered at elevation 152.2 metres. Borehole MTO 1 was terminated in a layer of clayey silt till which was encountered from elevation 179.2 metres. The clayey silt till at this borehole was interlayered with sandy silt and silty sand.

The clayey silt till had measured N values 11 to 110 blows per 0.3 metres and a natural water content of about 9 to 30 per cent. Previous testing indicated a generally low to intermediate plasticity clay with plastic and liquid limits of 17 to 28 and 28 to 35 per cent, respectively. The results of the Atterberg limit testing and a sample from borehole B indicated plastic and liquid limits of 12 and 22, respectively, as shown on Figure A-3 indicating a clay of low plasticity. Blow counts of approximately 50 to 95 blows per 0.3 metres were measured within the clayey silt till during dynamic cone penetration testing conducted adjacent to borehole MTO 1.

4.1.9 Bedrock

Borehole B was terminated in bedrock after exploring it for some 0.7 metres by mud rotary tricone drilling. The inferred surface of the bedrock is at elevation 149.8 metres.

4.2 Groundwater Conditions

Groundwater conditions were observed in the boreholes during drilling. Groundwater was encountered within the sand in borehole B at a depth of 29.36 metres below existing ground surface or at elevation 154.80 metres and within the silt at borehole 83-2 at a depth of 7.3 metres or at elevation 175.4 metres. Boreholes A and 83-1 remained dry during drilling. Groundwater levels were not indicated for boreholes 1 and 2.

The upper long-term groundwater level is expected to be at about elevation 181 metres based on the changes in soil colour and water content within the silty clay till and clayey silt till. A lower groundwater level near 155 metres is also inferred. Groundwater levels are subject to fluctuations due to seasonal and climatic variations or changes of flow levels in Puce River.

The water level in Puce River was at elevation 180.8 metres on July 23, 2004. The predicted 100 year high water level is 183.9 metres.


5.0 MISCELLANEOUS

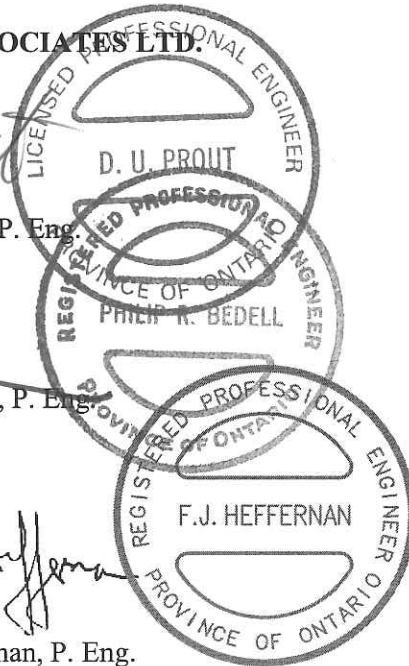
The investigation was carried out using equipment supplied and operated by Aardvark Drilling, Inc. (Aardvark). Aardvark is an Ontario Ministry of Environment licensed well contractor. Field operations were supervised by Mr. Mike Arthur under the direction of Mr. David J. Mitchell. All routine laboratory testing was conducted at Golder Associates' London laboratory. The consolidation test was conducted at Golder Associates' Mississauga laboratory. Both laboratories are accredited participants in the MTO's Soil and Aggregate Proficiency program and are certified for full quality testing of Types C and D Aggregates by the Canadian Council of Independent Laboratories. The Mississauga laboratory is registered in the specialty of Soil and Rock Including Testing for Foundation Engineering – Low and High Complexity. This report was written by Ms. Dirka U. Prout, P. Eng. under the direction of the Project Manager, Mr. Philip R. Bedell, P. Eng. The report was reviewed by Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor.

GOLDER ASSOCIATES LTD.


Dirka U. Prout, P. Eng.


Philip R. Bedell, P. Eng.
Principal


Fintan J. Heffernan, P. Eng.
MTO Designated Contact



DUP//PRB/FJH/jg

n:\active\2004\130000\041-130054 dillon - foundations - 401\reports\foundation reports\041-130054-0-3 puce river bridge\1224 (final) puce river bridge.doc

PART B

**FOUNDATION DESIGN REPORT
PUCE RIVER BRIDGE REPLACEMENT
HIGHWAY 401, SITE 6-083
FROM 1.5 KM WEST OF MANNING ROAD
EASTERLY TO 1.3 KM EAST OF PUCE ROAD
GWP 62-00-00, AGREEMENT NO. 3005-A-000393
MINISTRY OF TRANSPORTATION – SOUTHWESTERN REGION**

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the draft report provides our recommendations on the foundation aspects of the planning phase of the proposed replacement of the Puce Rive Bridge based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

It is understood that the existing Highway 401 will be widened and reconstructed between 1.5 kilometres west of Manning Road and 1.3 kilometres east of Puce Road. The work will also include rehabilitation of the Manning Road and Puce Road interchanges and the 6th and 9th Concession Roads along with reconstruction of the Pike Creek and Puce River Bridges within the project limits.

Based on the available information, it is understood that the existing bridge over the Puce River will be demolished and replaced by a single-span structure. According to the General Arrangement Drawing provided by Dillon, the new structure will be constructed using integral abutments, founded on piles and will have an approximate bridge deck elevation of 186.3 metres. Design information for the existing structure indicate that the existing single span, rigid frame structure was erected in 1951, and widened in 1967 is founded on three shallow strip footings located in the silty clay till near elevation 179.9 metres. The approximate elevation of the existing bridge deck is 185.7 metres. The bridge replacement will be concurrent with channel improvements at the bridge location.

6.2 Bridge Foundations

The subsoils encountered in the boreholes put down during this investigation typically consist of surficial topsoil and fill overlying loose sandy silt in localized areas which in turn overlies extensive deposits of stiff to very stiff, becoming firm to stiff below elevation 178 metres, silty clay or clayey silt till and silty clay till. The till in borehole B was underlain by sandy silt from elevation 172 metres, silty clay from elevation 170 metres, sand from elevation 155 metres, till from elevation 152 metres and terminated in inferred bedrock intercepted at below 149.8 metres.

6.3 Shallow Foundations

Subject to the configuration of the structure and design loads, the new bridge abutments may be founded on spread footings.

6.3.1 Axial Geotechnical Resistance

Based on the results of this investigation, spread footings may be founded at elevation 180 metres on the stiff to very stiff silty clay till. A factored geotechnical resistance of 300 kilopascals at the Ultimate Limit States (ULS) and a geotechnical resistance of 200 kilopascals at Serviceability Limit States (SLS) for an assumed 3 metre wide footing can be used for design purposes.

The settlement of these footings will be dependent on the footing size, configuration, and applied loads. It is anticipated that some 25 millimetres of total settlement would be experienced under full sustained load. Minor additional settlement of the existing embankments may occur due to consolidation of the founding soils as a result of the 0.6 metres grade raise required for the higher bridge deck. Differential settlement may also occur due to fill placement for the embankment widenings. However, widenings for the existing approach embankments could be constructed well in advance to reduce the footing settlements.

The geotechnical resistances provided are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the current Canadian Highway Bridge Design Code (CHBDC).

6.3.2 Resistance to Lateral Forces

Resistance to lateral forces/sliding resistance between the concrete spread footings and the subsoil should be calculated in accordance with Section 6.7.5 of the CHBDC. Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, the following angle of friction between the concrete and the founding soils, and corresponding coefficient of friction, $\tan \delta$, may be used:

Footings on silty clay till	angle of friction	28°
	$\tan \delta$	0.53

6.3.3 Frost Protection

All footings should be provided with a minimum of 1.2 metres of earth cover for frost protection purposes.

6.3.4 Construction Considerations

The founding soils are susceptible to softening upon exposure to water and the placement of a working slab will be required at the base of the excavation for the footing area. The cleaned excavation base should be inspected by qualified geotechnical personnel prior to placing the working slab. It is recommended that the footing excavation be carried out such that the final 0.5 metres of excavation is completed with the geotechnical personnel on site and the working slab be placed immediately after the excavation is inspected.

6.4 Deep Foundations

End bearing piles driven to practical refusal on either the hard clayey silt till at about elevation 151 metres or the underlying bedrock at about elevation 150 metres are considered suitable for support of the abutments and piers for the proposed structure. In the case of a design incorporating integral abutments, augering and placement of a corrugated steel pipe (CSP) liner around the upper 3 metres of the pile is required.

6.4.1 Geotechnical Axial Resistance

For design, the factored axial geotechnical resistance at ULS for HP 310 x 110 piles driven to refusal within the hard clayey silt till at about elevation 151 metres may be taken as 1,600 kilonewtons (kN) per pile. The till and underlying bedrock are considered to be unyielding layers and, therefore the geotechnical resistance at SLS does not apply. The piles should be equipped with suitable driving shoes such as the Titus 'H' Bearing Pile Point (Standard Model) or equivalent.

Provision should be made to re-tap the piles to confirm the set after adjacent piles have been driven in accordance with Special Provision 903S01.

6.4.2 Downdrag Load (Negative Skin Friction)

As discussed previously, some minor grade raise and inclining of the approach embankments is required to meet the bridge grade. The increased loading will cause some consolidation settlement of the underlying extensive clayey deposits as a result of increased vertical grades. The consolidation settlement is time-dependent and depending on the sequencing of construction may not completely occur during the construction period. That is, post-construction settlement of the clayey deposits may take place and settlement of the clayey soils relative to the piles will result in the development of negative skin friction acting on the piles. Therefore, negative skin friction or downdrag loads will need to be taken into account during design of the piles supporting the abutment additions. If the approach embankment grade raise is constructed well in

advance of the piling or if the additional fill loadings are minimized by the means of light weight fill the downdrag loads may be eliminated.

The magnitude of the downdrag load acting on a pile is a function of the adhesion (skin friction) that develops between the pile and the clay, the surface area of the pile within the clay deposit and the embankment loading. The load calculated in this manner is a nominal (unfactored) load. The structural engineer needs to multiply this load by a load factor of 1.25, as defined in the CHBDC, and include it as part of the load effects acting on the pile as described in the CHBDC. The negative skin friction is estimated to be 100 kN per pile.

6.4.3 Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The abutment piles will be driven through the cohesive tills with minor granular layers. The resistance to lateral loading may be based on the following assessed values:

SOIL TYPE	HORIZONTAL RESISTANCE VALUES (kN) PER PILE	
	Factored ULS	SLS
Clayey silt/silty clay till deposits	100	35

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor R as follows:

<i>Pile Spacing in Direction of Loading, d = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor R</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

6.4.4 Frost Protection

The pile caps should be provided with a minimum of 1.2 metres of soil cover for frost protection.

6.5 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments and associated retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. The following recommendations are made concerning the design of the abutments, in accordance with the CHBDC:

- Select, free-draining granular fill meeting the specifications of OPSS Granular A or Granular B but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 2501.00 and 2504.00.
- A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with CHBDC Figure 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06.
- The granular fill may be placed either in a zone with a width equal to at least 1.2 metres behind the back of the stem (Case i from Commentary on CHBDC Figure C6.9.1(I) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical extending up and back from the rear face of the footing (Case ii from Commentary on CHBDC Figure C6.9.1(I)).
- For Case i, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be assumed for granular fill:

Soil unit weight: 21 kN/m³

Coefficients of lateral earth pressure:

Active, K_a 0.33

At rest, K_o 0.50

- For Case ii, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	<u>GRANULAR A</u>	<u>GRANULAR B</u> (Type III)
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00.

6.6 Embankments

It is understood that in addition to widening the existing embankments in order to accommodate the widening of Highway 401, the height of the existing embankment fills will be increased by about 0.6 metres. Embankment side slopes formed no steeper than 2 horizontal to 1 vertical are considered suitable for this site. A Factor of Safety against deep seated failure of greater than 1.3 is available for the existing embankments constructed with suitable native or borrow materials.

Embankment settlements are estimated to be 10 millimetres in the area of the core of the existing embankment and 20 millimetres in the areas of the embankment widenings.

6.7 Excavations and Temporary Cut Slopes

Excavations for pile cap construction and/or spread footings will extend through existing fill and will encounter the silty clay till crust. Based on the subsurface conditions encountered in the boreholes, it is not likely that excavations will encounter the groundwater table. Temporary open cut slopes should be maintained no steeper than 1 horizontal to 1 vertical.

Some surficial water seepage into the excavations should be expected, and will be heavier during periods of sustained precipitation and/or high river flows. Pumping from filtered sumps located at the base of the excavations may be required to provide groundwater control during foundation excavations. Sumps should be maintained outside of the actual footing limits. Surface water runoff should be directed away from the excavations at all times. The appropriate Non Standard Special Provision (NSSP) should be included in the contract documents.


Where space is restricted and will not permit open cuts, a temporary roadway protection support system should be installed to support the sides of the excavation and permit the use of vertical cuts. The temporary support system could consist of soldier piles and lagging where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds or driven steel sheet piling. Support to the system could be in the form of struts and walers in the case of footing excavations or rakers and anchors in the case of roadway protection.


The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area line or point loads as well as the impact of sloping ground behind the system.

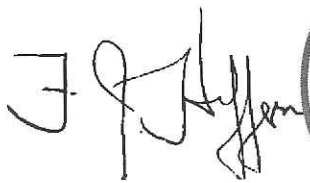
The temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 539S01. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 539S01.

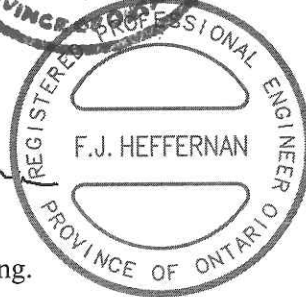
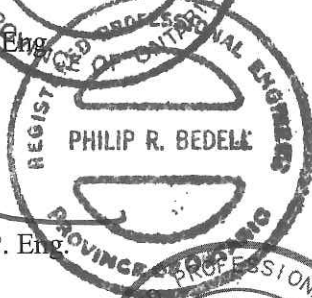
All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations For Construction Projects. The surficial topsoil, fill, and native sands or silts below the groundwater table at this site would be classified as Type 3 soils. The underlying cohesive deposits would be classified as Type 2.

GOLDER ASSOCIATES LTD.


Dirka U. Prout, P. Eng.


Philip R. Bedell, P. Eng.
Principal


Fintan J. Heffernan, P. Eng.
Designated MTO Contact



DUP/PRB/FJH/jg

N:\Active\2004\130000\041-130054 DILLON - FOUNDATIONS - 401\Reports\Foundation Reports\041-130054-0-3 Puce River Bridge\1224 (FINAL) Puce River Bridge.doc

LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole", on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

<i>AS</i>	auger sample
<i>CS</i>	chunk sample
<i>DO</i>	drive open
<i>DS</i>	Denison type sample
<i>FS</i>	foil sample
<i>RC</i>	rock core
<i>ST</i>	slotted tube
<i>TO</i>	thin-walled, open
<i>TP</i>	thin-walled, piston
<i>WS</i>	wash sample
<i>SS</i>	split spoon

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 0.3 m (12 in.).

Standard Penetration Resistance, N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 0.3 m (12 in.).

<i>WH</i>	sampler advanced by static weight-weight, hammer
<i>PH</i>	sampler advanced by hydraulic force
<i>PM</i>	sampler advanced by manual force

III. SOIL DESCRIPTION

(a) Cohesionless Soils

	"N" Blows/0.3 m or Blow/ft.
Relative Density	
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

	"Cu" = "Su"
Consistency	kPa psf.
Very soft	0 to 12 0 to 250
Soft	12 to 25 250 to 500
Firm	25 to 50 500 to 1000
Stiff	50 to 100 1000 to 2000
Very stiff	100 to 200 2000 to 4000
Hard	over 200 over 4000

IV. SOIL TESTS

<i>C</i>	consolidation test
<i>H</i>	hydrometer analysis
<i>M</i>	sieve analysis
<i>MH</i>	combined analysis, sieve and hydrometer ¹
<i>Q</i>	undrained triaxial ²
<i>R</i>	consolidated undrained triaxial ²
<i>S</i>	drained triaxial
<i>U</i>	unconfined compression
<i>V</i>	field vane test
<i>Chem</i>	chemical analysis

NOTES:

1. Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.
2. Undrained triaxial tests in which pore pressures are measured are shown as Q or R.

LIST OF SYMBOLS

I. GENERAL

π	= 3.1416
e	= base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of a
$\log_{10} a$ or $\log a$	logarithm of a to base 10
t	time
g	acceleration due to gravity
V	volume
W	weight
m	mass
M	moment
F	factor of safety

II. STRESS AND STRAIN

u	pore pressure
σ	normal stress
σ'	normal effective stress (σ is also used)
τ	shear stress
ϵ	linear strain
ϵ_{sy}	shear strain
ν	Poisson's ration (μ is also used)
E	modulus of linear deformation (Young's modulus)
G	modulus of shear deformation
K	modulus of compressibility
η	coefficient of viscosity

III. SOIL PROPERTIES

(a) Unit weight

γ	unit weight of soil (bulk density)
γ_s	unit weight of solid particles
γ_w	unit weight of water
γ_d	unit dry weight of soil (dry density)
γ'	unit weight of submerged soil
G_s	specific gravity of solid particles $G_s = \gamma_s / \gamma_w$
e	void ratio
n	porosity
w	water content
S_r	degree of saturation

(b) Consistency

w_L	liquid limit
w_P	plastic limit
I_P	plasticity index
w_S	shrinkage limit
I_L	liquidity index = $(w - w_P) / I_P$
I_C	consistency index = $(w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
D_r	relative density = $(e_{max} - e) / (e_{max} - e_{min})$

(c) Permeability

h	hydraulic head or potential
q	rate of discharge
v	velocity of flow
i	hydraulic gradient
κ	coefficient of permeability
j	seepage force per unit volume

(d) Consolidation (one-dimensional)

m_v	coefficient of volume change = $-\Delta e / (1+e) \Delta \sigma'$
C_c	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
c_v	coefficient of consolidation
T_F	time factor = $c_v t / d^2$ (d , drainage path)
U	degree of consolidation

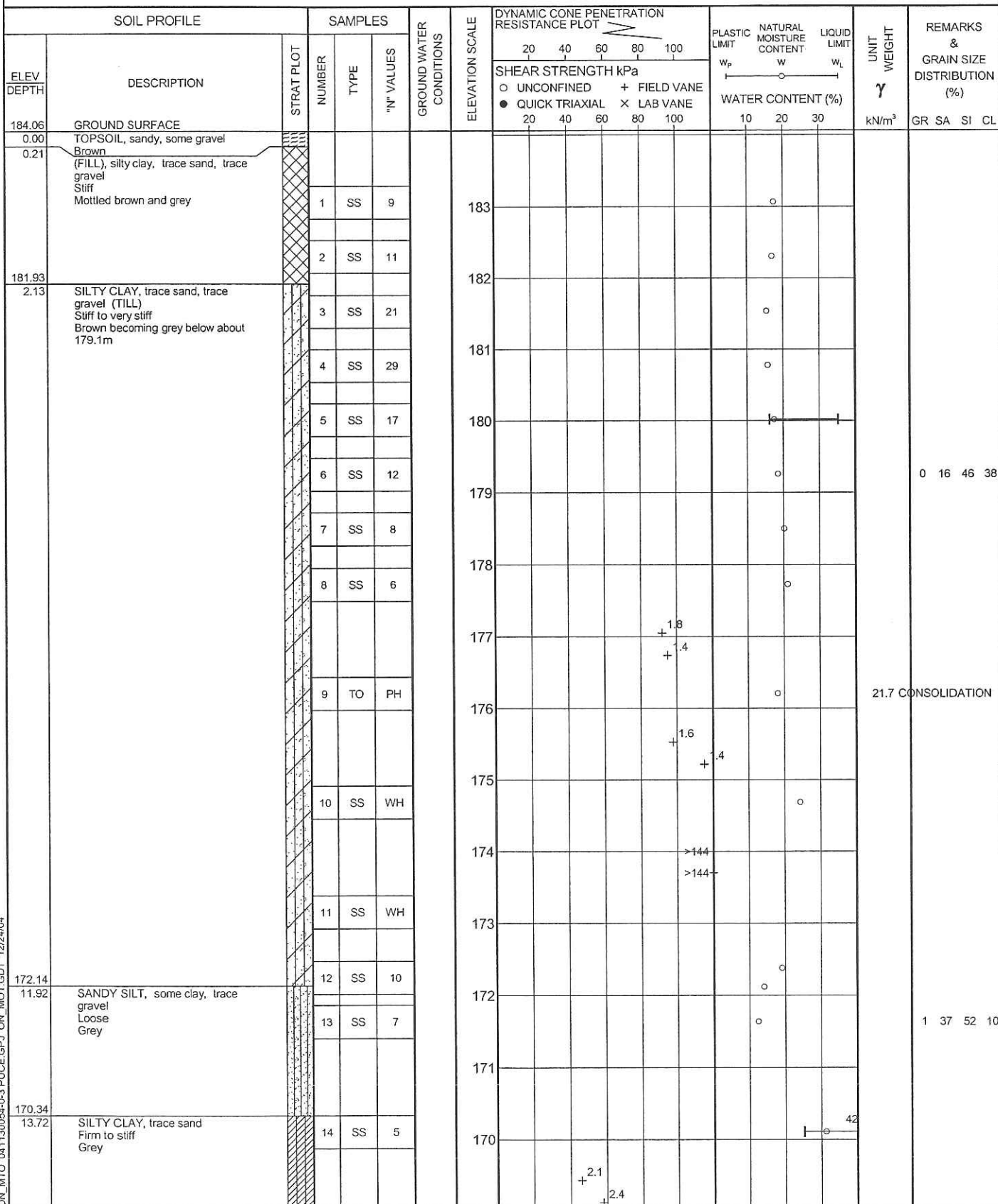
(e) Shear strength

τ_f	shear strength	in terms of effective stress $\tau_f = c' + \sigma' \tan \phi$
c'	effective cohesion intercept	
ϕ'	effective angle of shearing resistance, or friction	
S_u	apparent cohesion*	
ϕ_u	apparent angle of shearing resistance, or friction	in terms of total stress $\tau_f = c_u + \sigma \tan \phi_u$
μ	coefficient of friction	
S_t	sensitivity	

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = S_u$ is taken as half the undrained compressive strength.

+³, ×³: Numbers refer to Sensitivity ○³% STRAIN AT FAILURE

PROJECT 041-1130054-0-3		RECORD OF BOREHOLE No B		1 OF 3	METRIC
G.W.P. 62-00-00	LOCATION N 4677749.4 ; E 280529.2	ORIGINATED BY MR			
DIST 1 HWY 401	BOREHOLE TYPE POWER AUGER (HOLLOW STEM) / TRI CONE MUD ROTARY	COMPILED BY BG			
DATUM GEODETIC	DATE July 23, 2004 - July 27, 2004	CHECKED BY			



ON MTO 041130054-0-3 PUCE.GPJ ON MOT.GDT 12/24/04

Continued Next Page

+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

METRIC

PROJECT 041-1130054-0-3

G.W.P. 62-00-00

LOCATION N 4677749.4 :E 280529.2

ORIGINATED BY MR

DIST 1 HWY 401

BOREHOLE TYPE POWER AUGER (HOLLOW STEM) / TRI CONE MUD ROTARY

COMPILED BY BG

DATUM GEODETIC

DATE July 23, 2004 - July 27, 2004

CHECKED BY

[illegible]

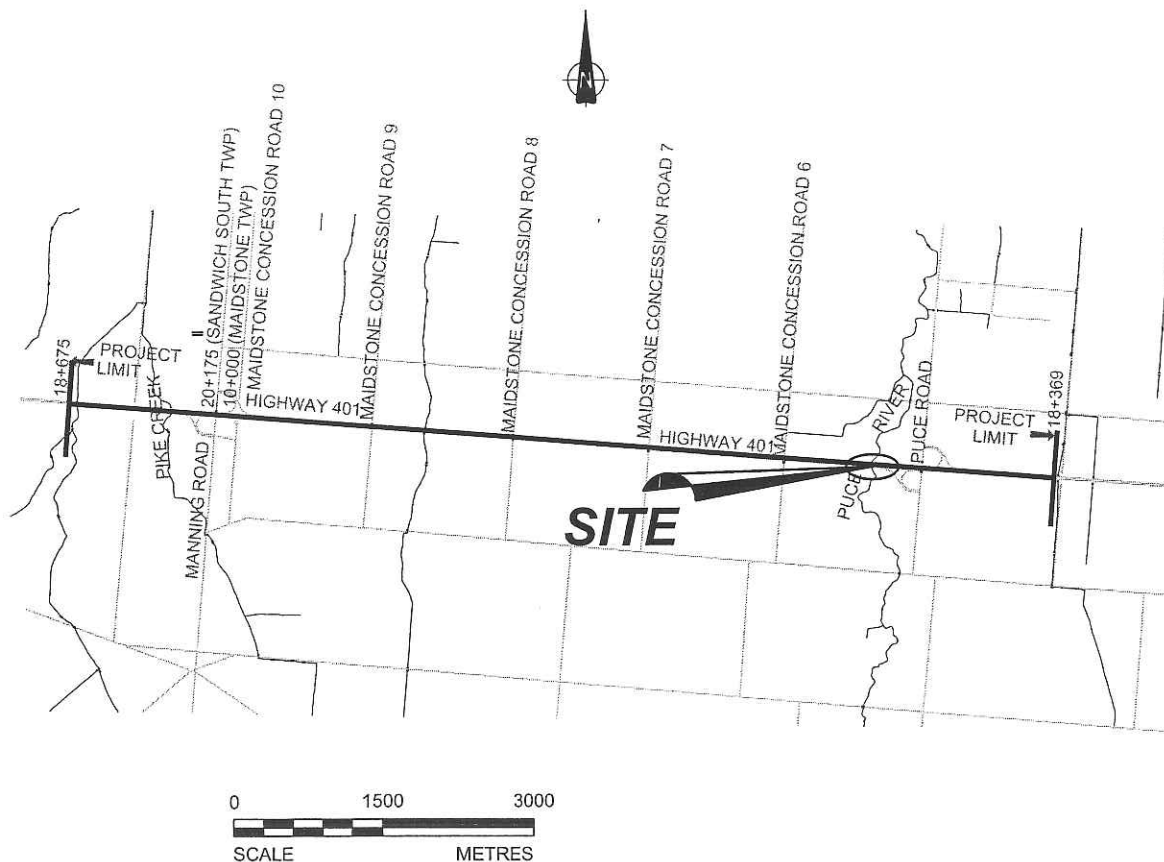
Continued Next Page


+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

PROJECT <u>041-1130054-0-3</u>		RECORD OF BOREHOLE No B		3 OF 3	METRIC
G.W.P. <u>62-00-00</u>	LOCATION <u>N 4677749.4 E 280529.2</u>	ORIGINATED BY <u>MR</u>			
DIST <u>1</u> HWY <u>401</u>	BOREHOLE TYPE <u>POWER AUGER (HOLLOW STEM) / TRI CONE MUD ROTARY</u>	COMPILED BY <u>BG</u>			
DATUM <u>GEODETIC</u>	DATE <u>July 23, 2004 - July 27, 2004</u>	CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID 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					W _p	W	W _L		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE									
	SAND, fine, trace silt, trace gravel Dense, Grey																
			21	SS	36												
152.21							153										
31.85	CLAYEY SILT, trace sand, trace gravel (TILL), with cobbles and boulders Hard Grey																

ON_MTO_041130054-0-3_PUOE.GPJ ON_MOT.GDT 12/24/04



PROJECT		PUCE RIVER BRIDGE GWP 62-00-00 HIGHWAY 401			
TITLE		KEY PLAN			
 Golder Associates LONDON, ONTARIO		PROJECT No. 041-1300540-3		FILE No. 041-130054-0-3D000	
		CADD	BG	DEC. 2004	SCALE AS SHOWN
		CHECK	DP	NOV. 2004	REV. 0
		FIGURE 1			

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST 1 HWY. 401
CONT. No. 2005-3001
WP No. 62-00-00



PUCE RIVER BRIDGE

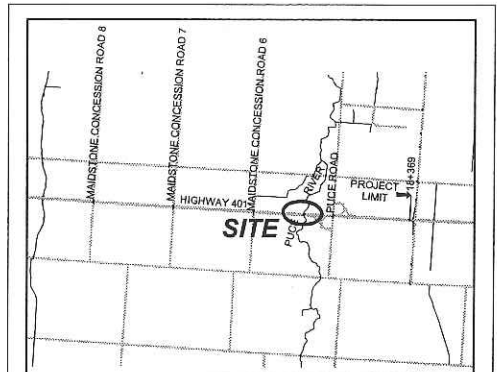
SHEET

BOREHOLE LOCATIONS



Golder Associates Ltd.
LONDON, ONTARIO, CANADA

REFERENCE
DRAWING SUPPLIED BY DILLON CONSULTING
ENTITLED: PUCE RIVER BRIDGE
GENERAL ARRANGEMENT
GWP - 62-00-00



KEY PLAN

LEGEND

- Borehole
- Previous Borehole by Others
- Seal
- Piezometer
- N Blows/0.3m (Std. Pen. Test, 475 j/blow)

No.	ELEVATION (metres)	CO-ORDINATES	
		NORTH	EAST
A	184.51	4677725.3	280528.2
B	184.06	4677749.4	280529.2
MT0 1	183.73	4677746.0	280528.0
MT0 2	183.18	4677731.0	280530.0
83-1	184.62	4677725.0	280483.0
83-2	182.70	4677742.0	280482.0

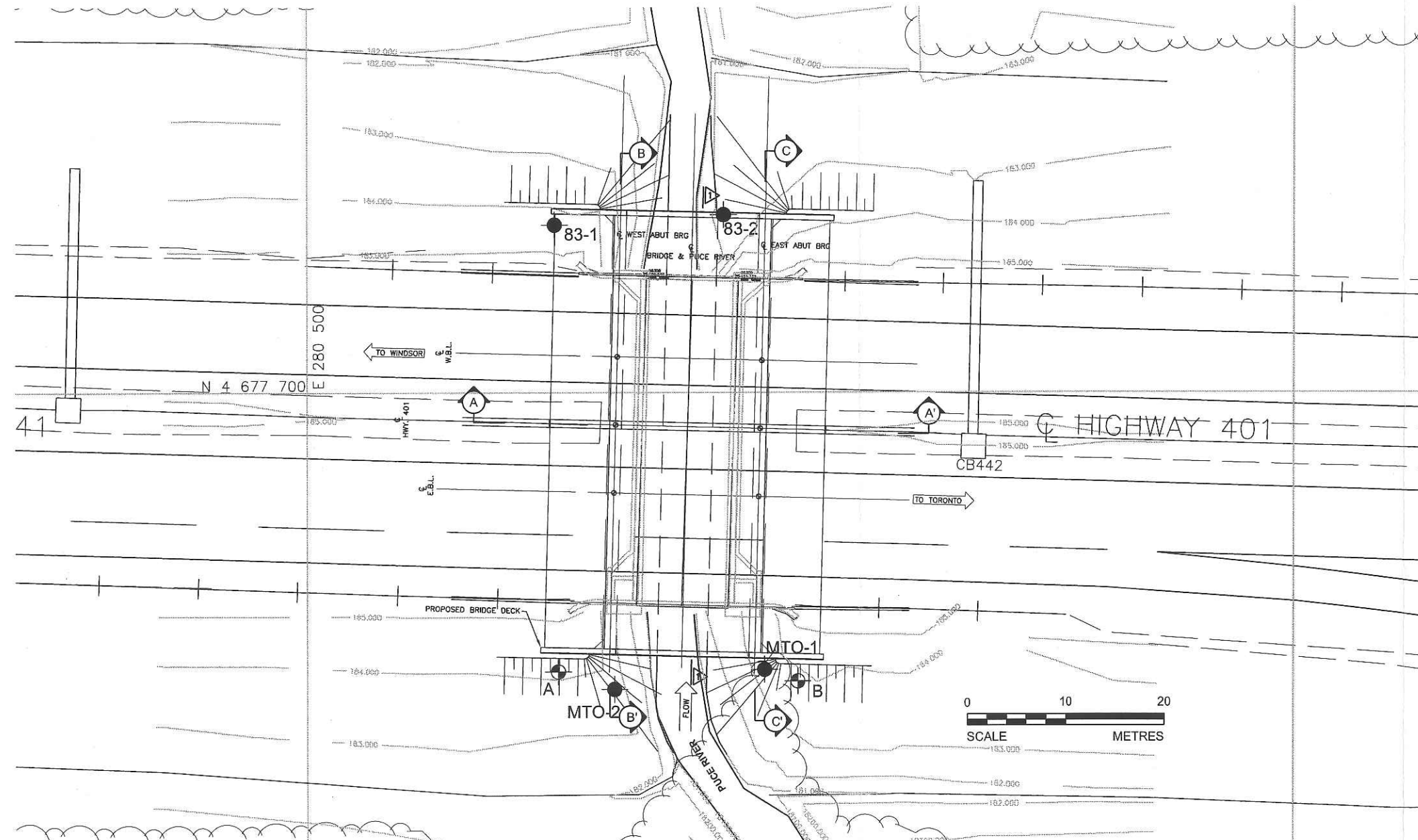
NOTES

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

NO.	DATE	BY	REVISION

Geocres No. 40J2-65

HWY. No.	401	PROJECT NO.	041-130054-0-3
SUBM'D.	-	CHKD.	-
DRAWN:	BG	CHKD.	DP
DATE:	DEC. 2004	APPD.	-
DWG.	1		





PUCE RIVER BRIDGE

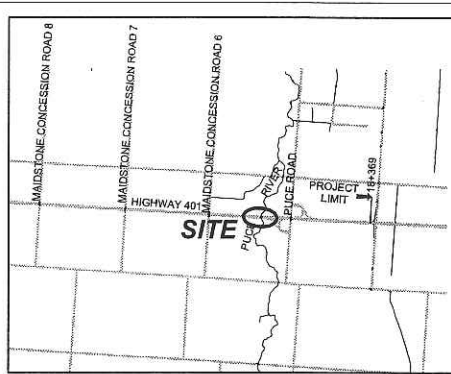
SHEET

SOIL STRATA



Golder Associates Ltd.
LONDON, ONTARIO, CANADA

REFERENCE
DRAWING SUPPLIED BY DILLON CONSULTING
ENTITLED: PUCE RIVER BRIDGE
GENERAL ARRANGEMENT
GWP - 62-00-00



KEY PLAN

LEGEND

- Borehole
- Previous Borehole by Others
- Seal
- Piezometer
- N Blows/0.3m (Std. Pen. Test, 475 j/blow)
- WL in piezometer
- WL during drilling
- DRY Borehole dry during drilling

No.	ELEVATION (metres)	CO-ORDINATES	
		NORTH	EAST
A	184.51	4677725.3	280528.2
B	184.06	4677749.4	280529.2
MTO 1	183.73	4677746.0	280528.0
MTO 2	183.18	4677731.0	280530.0
83-1	184.62	4677725.0	280483.0
83-2	182.70	4677742.0	280482.0

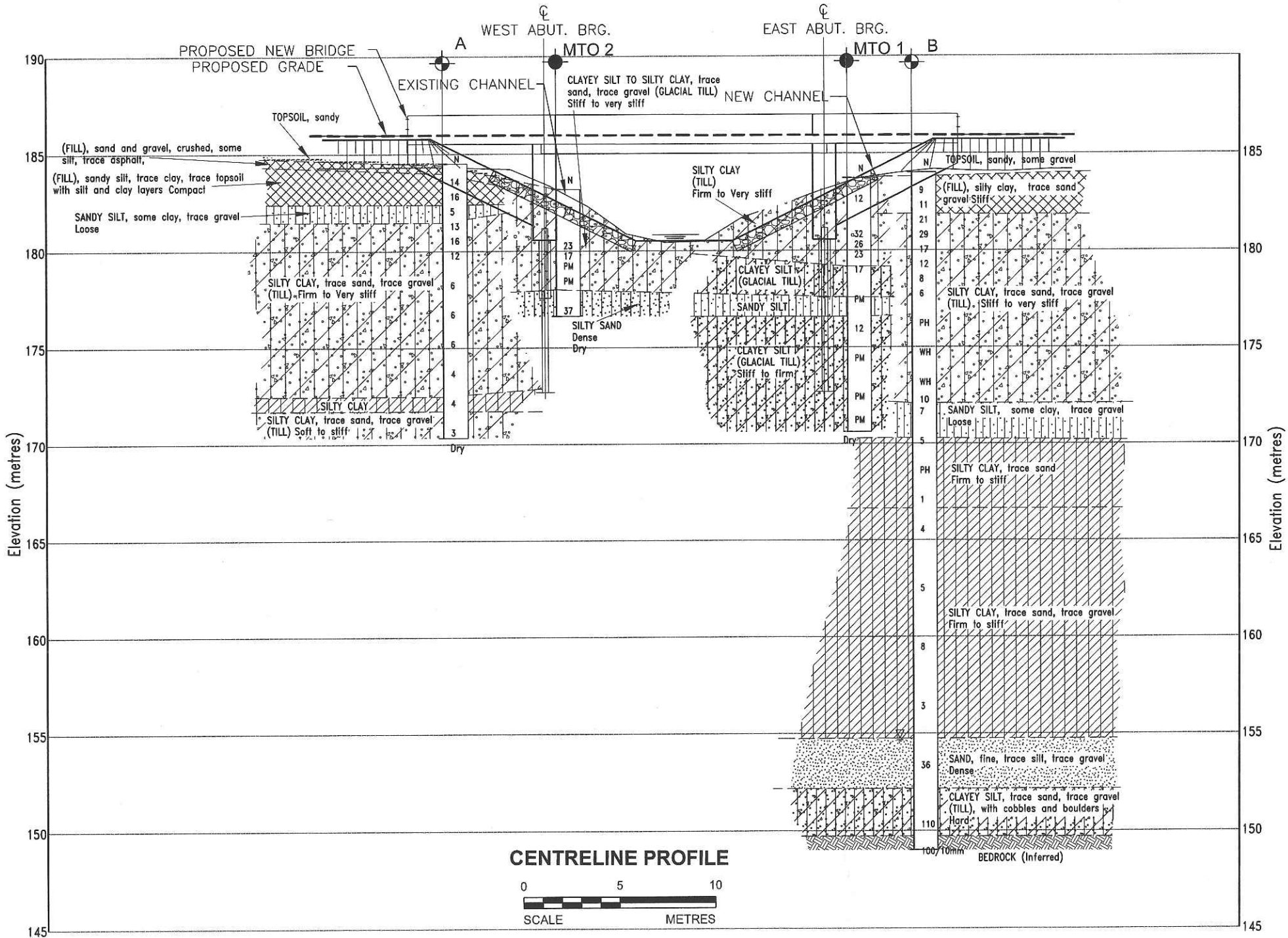
NOTES

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

NO.	DATE	BY	REVISION

Geocres No. 40J2-65

HWY. No.	401	PROJECT NO.	041-130054-0-3
SUBM'D.	-	CHKD.	-
DRAWN:	BG	CHKD.	DP
		APPD.	
		DWG.	2




CENTRELINE PROFILE

0 5 10
SCALE METRES

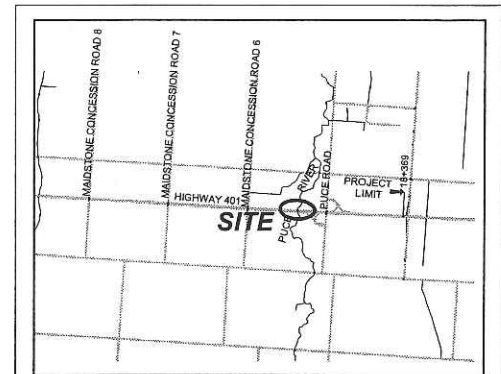
DIST HWY. 401
CONT. No.
WP No. 62-00-00

PUCE RIVER BRIDGE


SOIL STRATA


Golder Associates Ltd.
LONDON, ONTARIO, CANADA


REFERENCE
DRAWING SUPPLIED BY DILLON CONSULTING
ENTITLED: PUCE RIVER BRIDGE
GENERAL ARRANGEMENT
GWP - 62-00-00





LEGEND


 Borehole


 Previous Borehole by Others


 Seal

 Piezometer

 Blows/0.3m (Std. Pen. Test, 475 j/blow)

 WL in piezometer

 WL during drilling

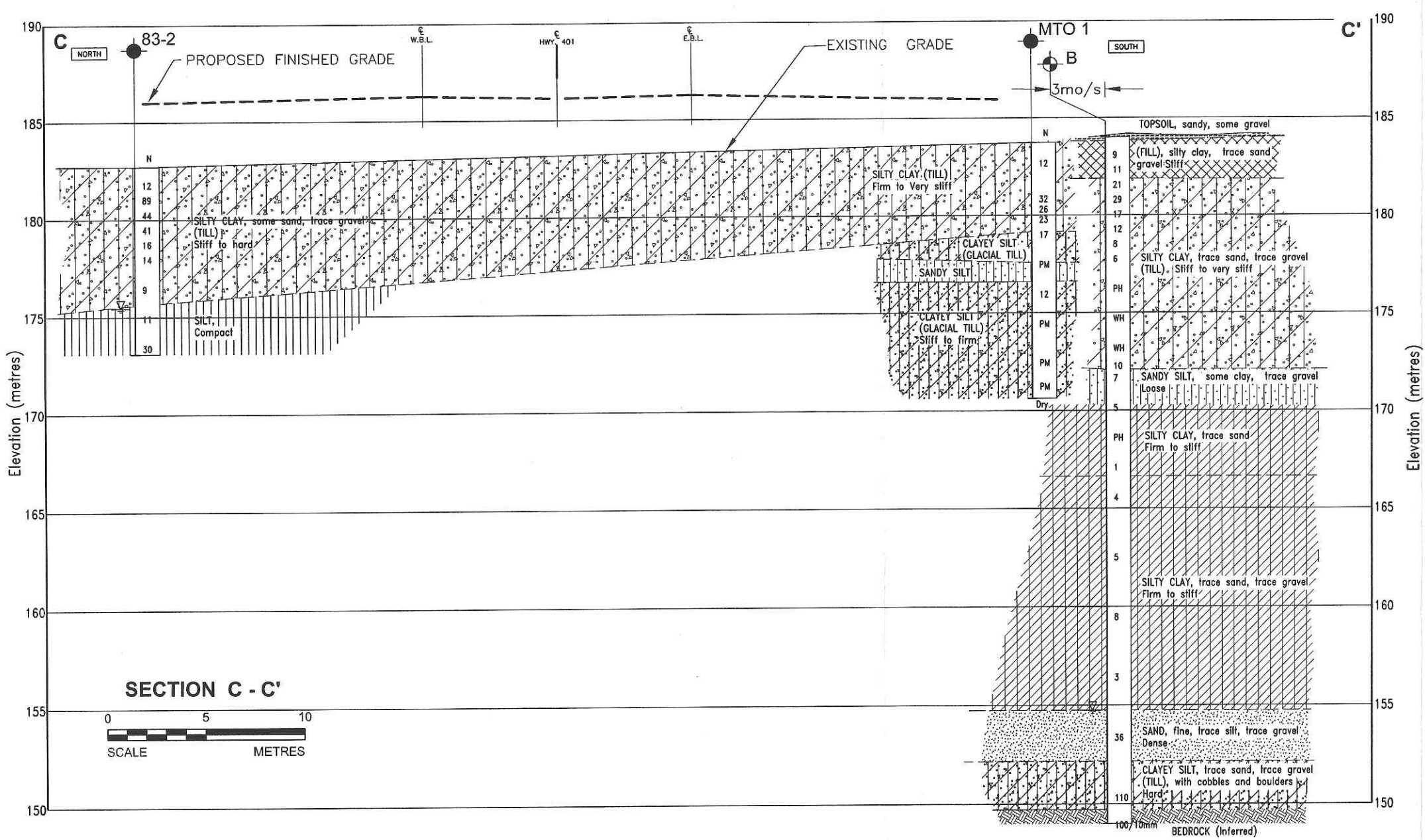
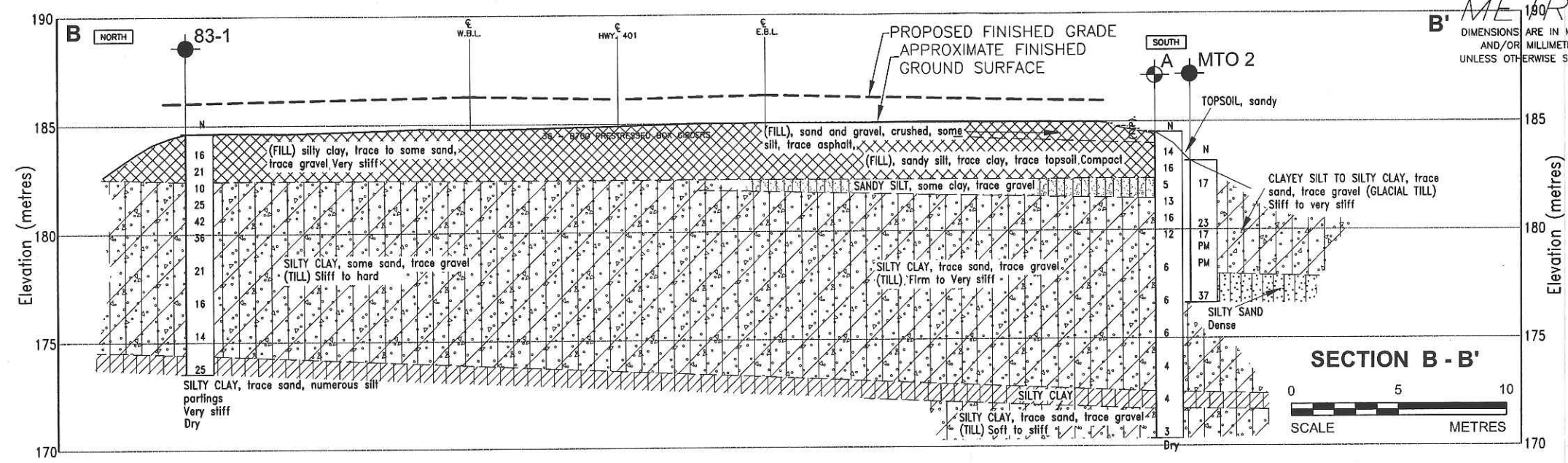
 DRY Borehole dry during drilling

No.	ELEVATION (metres)	CO-ORDINATES	
		NORTH	EAST
A	184.51	4677725.3	280528.2
B	184.06	4677749.4	280529.2
MTO 1	183.73	4677746.0	280528.0
MTO 2	183.18	4677731.0	280530.0
83-1	184.62	4677725.0	280483.0
83-2	182.70	4677742.0	280482.0

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

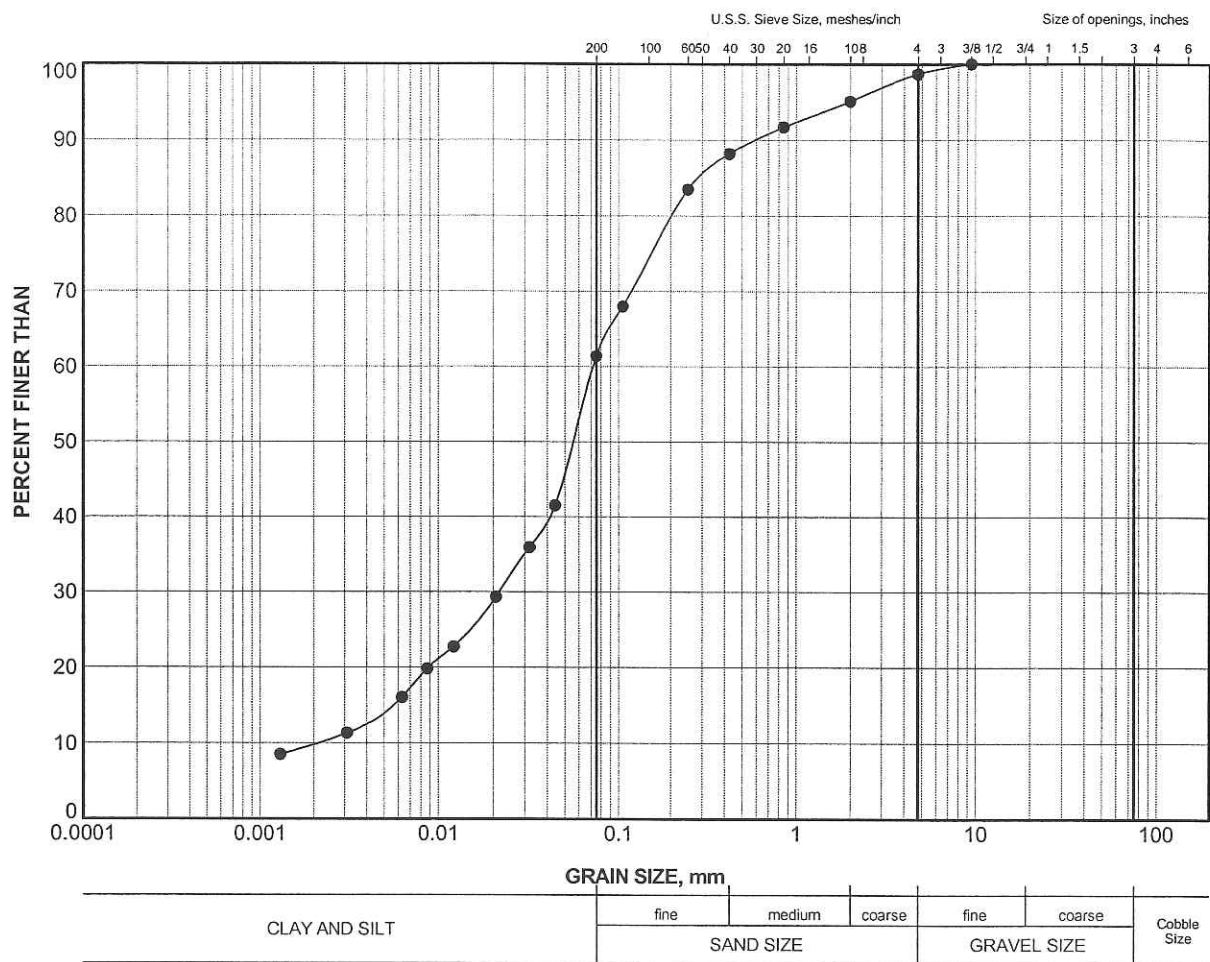
NO.	DATE	BY	REVISION
Geocres No. 40J2-65			
HWY. No.	401	PROJECT NO.	041-130054-0-3
SUBM'D.	-	CHKD.	-
DATE:	DEC. 2004	DATE:	DEC. 2004
DRAWN:	BC	CHKD.	DP
APPD.		APPD.	
DWG.	3		

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN




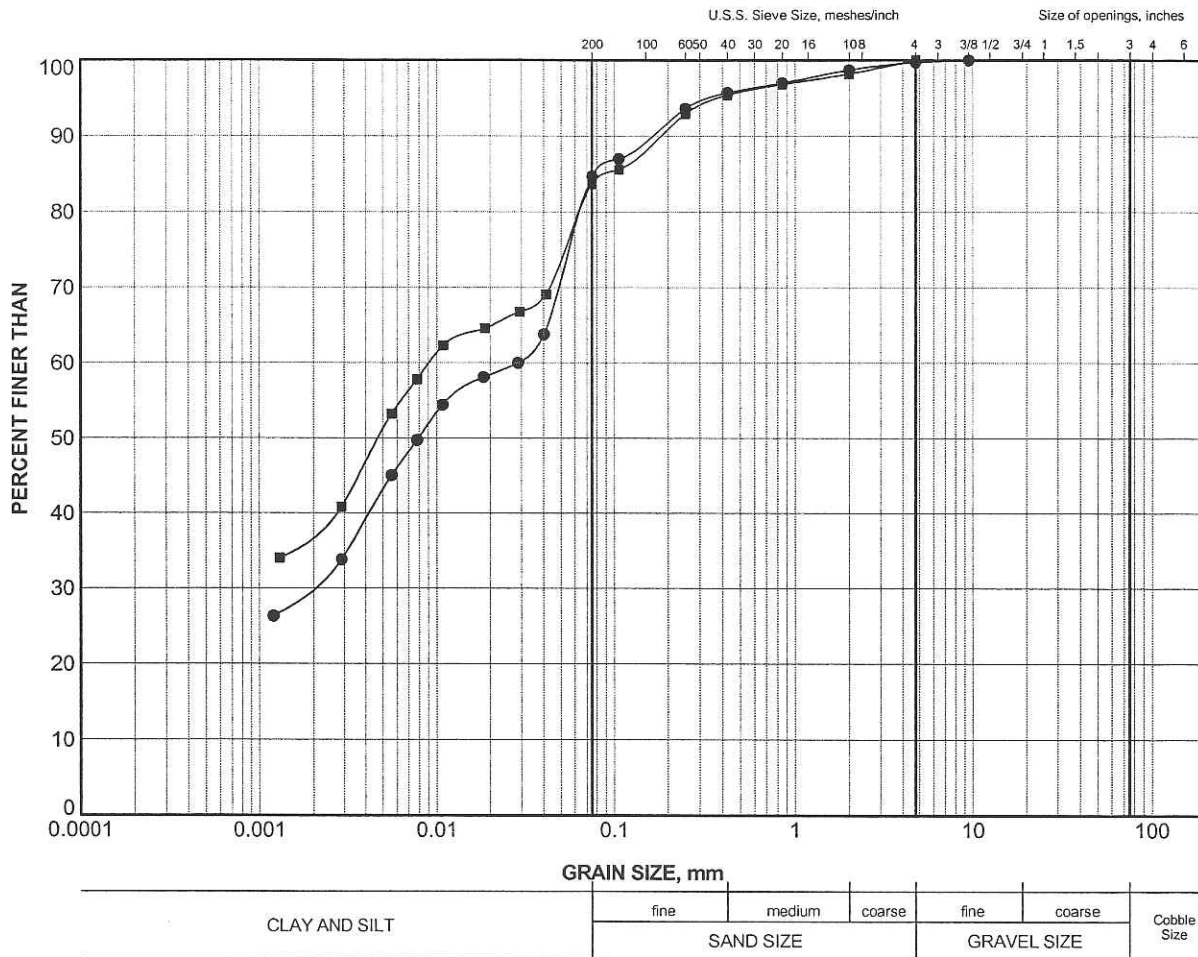
APPENDIX A

LABORATORY TEST DATA (FIGURES A-1 TO A-4)




LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	B	13	172.3

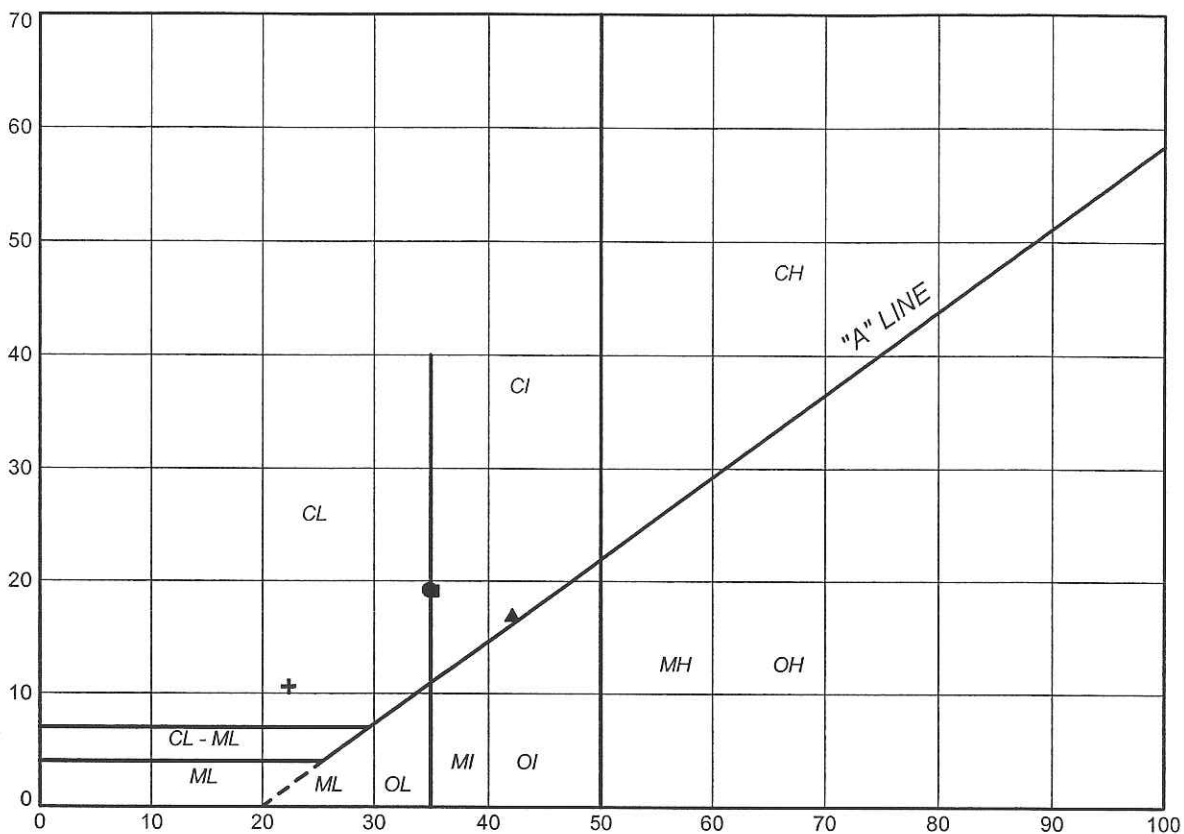
PROJECT		PUCE RIVER BRIDGE GWP 62-00-00 HIGHWAY 401	
TITLE		GRAIN SIZE DISTRIBUTION SANDY SILT	
 Golder Associates LONDON, ONTARIO	PROJECT No. 041-1130054-0-3		FILE No. 041130054Puce.GPJ
	DRAWN	BG	Nov 10/04
	CHECK	DP	Dec. 24/04
		SCALE	N/A
		REV.	
FIGURE A-1			



LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	A	8	176.7
■	B	6	179.3

PROJECT		PUCE RIVER BRIDGE GWP 62-00-00 HIGHWAY 401			
TITLE		GRAIN SIZE DISTRIBUTION SILTY CLAY TILL			
PROJECT No. 041-1130054-0-3		FILE No. 041130054Puce.GPJ			
DRAWN BG		Nov 10/04		SCALE N/A	
CHECK DP		Dec. 24/04		REV.	
 Golder Associates LONDON, ONTARIO		FIGURE A-2			

PLASTICITY INDEX (Percent)



LIQUID LIMIT (Percent)

SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)	LL(%)	PL(%)	PI
●	A	9	175.4	34.9	15.7	19.2
■	B	5	180.3	35.2	16.1	19.1
▲	B	14	170.3	42.1	25.1	17.0
+	B	22	150.5	22.4	11.8	10.6


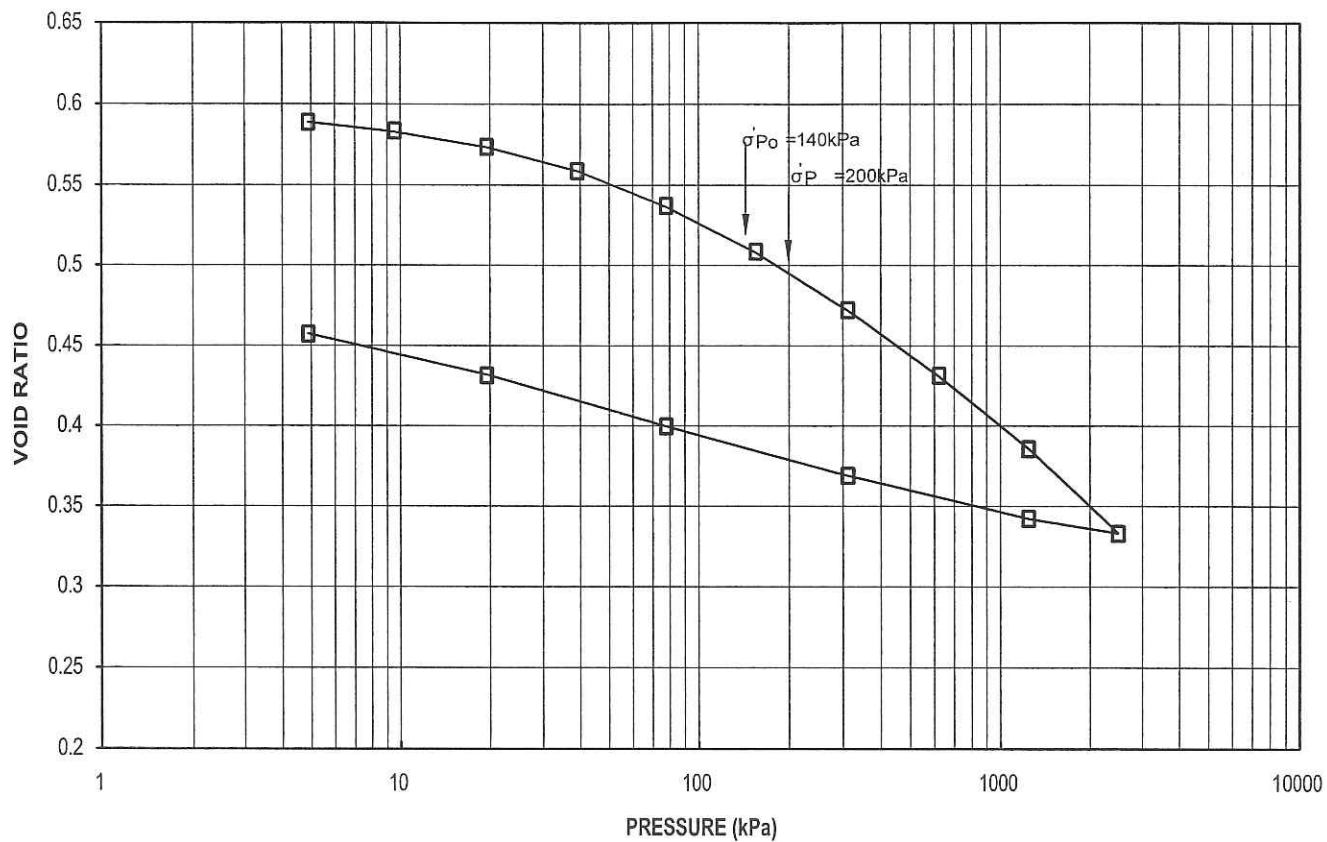
PROJECT		PUCE RIVER BRIDGE GWP 62-00-00 HIGHWAY 401	
TITLE		PLASTICITY CHART	
 Golder Associates LONDON, ONTARIO		PROJECT No. 041-1130054-0-3 DRAWN BG Nov 12/04 CHECK DP Dec 24/04	FILE No. 041130054Puce.GPJ SCALE N/A REV.

FIGURE A-3

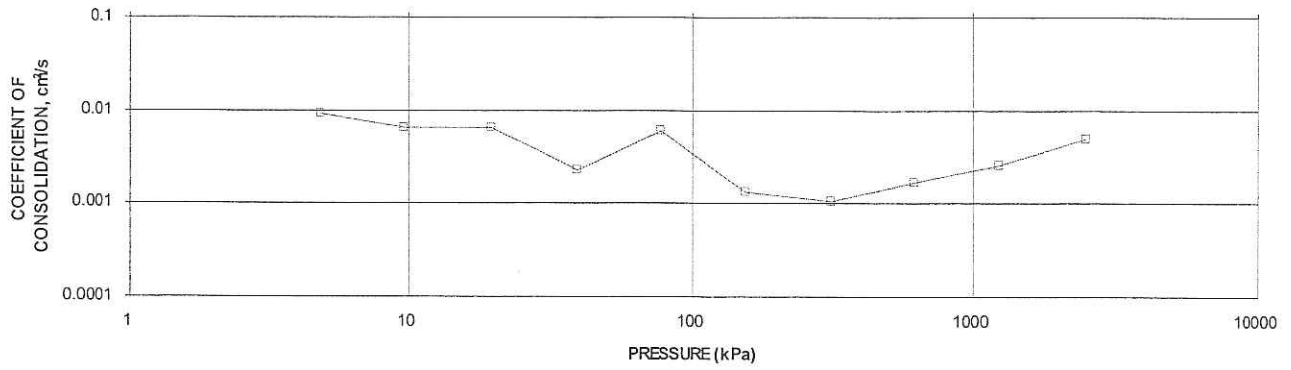


BOREHOLE B, SAMPLE 9
ELEV. 176.2m

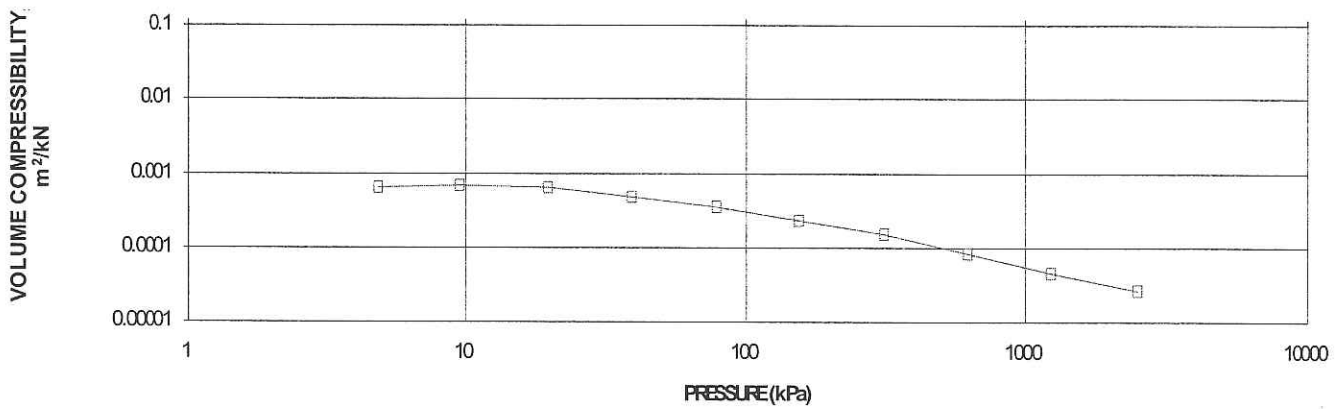
PROJECT		PUCE RIVER BRIDGE GWP 62-00-00 HIGHWAY 401	
TITLE		CONSOLIDATION TEST VOID RATIO Vs. LOG PRESSURE	
PROJECT No. 041130054-0-3		FILE No. 041130054-0-3	
CADD BG NOV. 10/04		SCALE AS SHOWN REV. 0	
CHECK DP DEC. 22/04		FIGURE A-4	



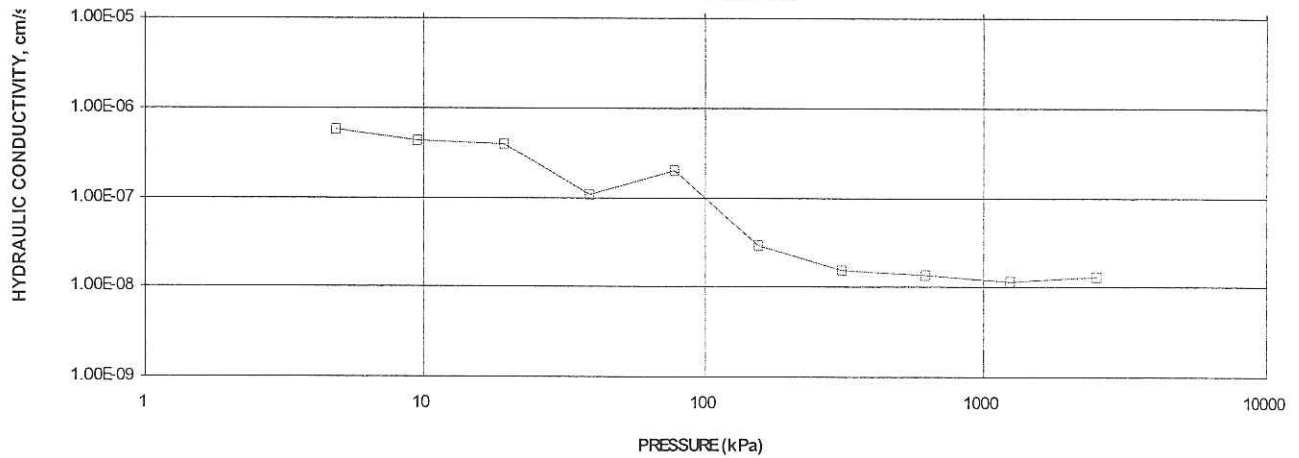
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BHB SA 9




CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BHB SA 9



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BHB SA 9



PROJECT		PUCE RIVER BRIDGE GWP 62-00-00 HIGHWAY 401	
TITLE		OEDOMETER CONSOLIDATION SUMMARY	
PROJECT No. 041130054-0-3		FILE No. 041130054-0-3	
CADD	BG	NOV. 10/04	SCALE AS SHOWN
CHECK	DP	DEC. 22/04	REV. 0
 Golder Associates LONDON, ONTARIO		FIGURE A-5	

APPENDIX B

RECORDS OF BOREHOLES FROM PREVIOUS INVESTIGATIONS

RECORD OF BOREHOLE No 83-1

1 of 1 METRIC

C.W.P. 80-00-00

LOCATION

Co-ords. 4 877 717 N; 240 535 E.

ORIGINATED BY MR

DIST 3.2

HWY 401

BOREHOLE TYPE

Continuous Flight Solid Stem Augers

COMPILED BY GD

DATUM Geodetic

DATE

May 11, 2004

CHECKED BY MRJ

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					
184.62	Ground Level															
0.00	Silty clay, trace to some sand, trace of gravel Very Stiff Brown (Fill)		1	SS	16											
183.32																
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 (Fill)		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											
174.43																
10.20	Silty clay, trace of sand, with numerous thin distorted lenses and partings of silt															
173.62	Very Stiff Grey		10	SS	25											
11.10	End of Borehole															
	Borehole dry on completion of drilling • Penetrometer Test															

RECORD OF BOREHOLE No 83-2

1 of 1 METRIC

C.W.P. 40-00-00 LOCATION Co. Rd. 4 625 718 102 200 M. E. ORIGINATED BY MR
 DIST 10 HWY 401 BOREHOLE TYPE Continuous Flight Auger Drilling COMPILED BY GD
 DATUM Canadian DATE Nov 12, 2002 CHECKED BY NNA

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 3 GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100					
182.70	Ground Level															
0.00	Silty clay, some sand, trace of gravel Stiff to hard Brown (Till)		1	SS 13		182										
			2	SS 40		181										
			3	SS 41		180										2 13 41 38
			4	SS 41		179										
	Very Stiff to Stiff Grey		5	SS 16		178										
			6	SS 14		177										
			7	SS 9		176										2 15 44 39
172.80	Silt Compact Grey Saturated		8	SS 11		175										
7.10			9	SS 10		174										
173.10	End of Borehole															
3.60																

ON_MOT 017673C.GPJ ON_MOT.GDT 11/10/2002 12:55:02 PM

Numbers refer to
 Sensitivity
 20
 15-0-5 (%) STRAIN AT FAILURE
 10

FORM NO. 10-1-1-10
11-10-10

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAY - DISTRICT
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

100 64-2-70 LOCATION Pine River crossing of Hwy 401, Southwest of Arden ESTIMATED BY J.D.
 W.P. 607-16 BORING DATE July 15, 1965 COMPILED BY L.P.
 DATUM Prostatic BOREHOLE TYPE Washbourn CHECKED BY H.D.

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLING NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT					LIQUID LIMIT PLASTIC LIMIT WATER CONTENT	WATER CONTENT %	BULK DENSITY	REMARKS
						20	40	60	80	100				
(183.73)	602.8 0.0													
	Silty clay, some sand, trace gravel (glacial till) stiff to hard, grey-brown.	1	SS	11										
		2	SS	12										
		3	SS	26										
		4	SS	27										
(179.18) (4.55)	587.8 15.0	5	SS	17										
	Clayey silt, some sand trace gravel, (glacial till) stiff to firm.													
(177.58) (6.15)	582.8 20.2	6	TM	PH										
	Layer of sandy silt.													
(170.58) (7.15)	579.3 23.5	7	SS	12										
		8	TM	PH										
		9	TM	PH										
(171.23) (12.50)	571.2 41.0	10	TM	PH										
	Clayey silt, some sand, trace gravel, (glacial till) stiff to firm.													
(170.63) (13.10)	559.8 41.0													
	End of borehole.													

() Metric depths and elevations are bracketed.

1917-18
 1918-19

OFFICE OF THE ATTORNEY GENERAL

RECORD OF BOREHOLE NO. 2										FOUNDATION SECTION						
MATERIALS & TESTING DIVISION				LOCATION Pine River Crossing of Hwy 401, Southwest of Bellevue				ORIGINATED BY L.P.								
JOB 01-75				BORING DATE July 16, 1965				COMPILED BY J.P.								
W.P. 691-62				BOREHOLE TYPE Washborings				CHECKED BY H.J. [initials]								
SOIL SAMPLE		SAMPLES		DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — % PLASTIC LIMIT — % WATER CONTENT — %		DISTINCTION NO. P.T.F.	REMARKS				
ELEV. DEPTH	DESCRIPTION	SIGNAL PLAT	NUMBER TYPE BLOWS / FOOT	ELEV. SCALE	1000	2000	3000	4000	5000	40			20	30		
(183.18)	501.0	Ground Surface														
	0.0	Clayey silt to silty clay some sand, trace gravel (clastic till) stiff to very stiff grey brown.			500											
(179.53) (3.65)	549.0 12.0	Grey			370											Salisbury 435 CL 432
(177.98) (5.20)	574.0 27.0	Silty sand - grey, dense.			580											Salisbury 435 CL 432
(176.83) (8.35)	579.0 21.5	End of borings.														Dr 254733 SL 4CL 255

() Metric depths and elevations are bracketed.

APPENDIX C
SITE PHOTOGRAPHS

SITE PHOTOGRAPHS



Photo 1: Southeast Wing Wall



Photo 2: Bridge Deck Looking North