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FOUNDATION INVESTIGATION AND DESIGN REPORT
PIKE CREEK BRIDGE REPLACEMENT
HIGHWAY 401, SITE 6-075
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

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FOUNDATION INVESTIGATION REPORT
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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 Pike Creek bridge by drilling boreholes, carrying out in-situ tests and laboratory tests 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 centreline 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 replacement of the Pike Creek 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 29 and August 24, 2004 at 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:

BOREHOLE	LOCATIONS (m) Northing	LOCATIONS (m) Easting	GROUND SURFACE ELEVATION (m)	BOREHOLE DEPTH (m)
A	4678012.7	272577.6	185.62	12.65
B	4678013.3	272558.3	185.54	39.68

The soil stratigraphy encountered in the boreholes is shown on the attached Record of Borehole sheets and on drawings 2 and 3.

The boreholes were advanced using track 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. In situ vane testing was carried out, where feasible, within the cohesive strata. Groundwater conditions were observed in the 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. 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, 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 are shown on Drawing 1, attached.

In addition, the results of boreholes from a previous investigation carried out by Peto MacCallum Ltd. for the proposed widening of the existing Pike Creek Bridge has been included in this report. The boreholes are:

- Boreholes 75-1, 75-2, 75-3 and 75-4, Geocres No. 40J2-55, Report No. 01TF073B entitled "Foundation Investigation Report for Widening of Pike Creek Bridge, G.W.P. 60-00-00, Site 6-75, Highway 401, Town of Tecumseh, Ontario", dated November 2002.

The four boreholes were drilled during the period from May 6 to 14, 2002 and advanced to depths of 6.6 to 14.2 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 records of the previous boreholes are provided in Appendix B.

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 Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and may 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 Pike Creek bridge replacement encountered topsoil and fill materials underlain by layers of clayey silt till, silty clay till, sandy silt till, sand and 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 230 millimetres thick. A buried topsoil layer some 200 millimetres thick was intercepted below the fill at elevation 183.5 metres at the base of the fill layer in borehole 75-4.

Beneath the topsoil in borehole A, and from the ground surface at boreholes 75-2, 75-3 and 75-4 fill layers between 1.0 to 2.5 metres thick were encountered. The fill in borehole A consisted of silt with some clay. The fill at the previous boreholes was composed of silty clay with topsoil inclusions being noted in the fill in borehole 75-4. The fill had measured N values, as determined in the standard penetration testing, ranging from 8 to 20 blows per 0.3 metres of penetration. The water contents of the fill varied between 14 and 20 per cent.

4.1.2 Clayey Silt Till

Beneath the topsoil in borehole B a layer of clayey silt till was encountered. The layer was some 3.1 metres thick and encountered at elevation 173.7 metres.

The upper very stiff to hard clayey silt till had measured N values between 21 and 35 blows per 0.3 metres. The clayey silt had natural water contents ranging from 11 to 15 per cent with an average water content of about 13 per cent.

4.1.3 Silty Clay Till

The silty clay till was intercepted between elevations 182.2 to 184.8 metres in all of the boreholes. Where fully penetrated in boreholes A, B and 75-3, the layers were 9.3 to 24.8 metres thick. Boreholes 75-1 and 75-4 were terminated in the silty clay till after exploring for some 5.0 to 13.2 metres.

The upper stiff to very stiff crust of the silty clay till was some 2.1 and 1.7 metres thick in boreholes A and B, respectively. In these boreholes, the upper crust had measured N values between 12 and 22 blows per 0.3 metres with an average N value of about 18 blows per 0.3 metres. Standard penetration test results in the previous boreholes ranged from 12 to 42 blows per 0.3 metres above approximate elevation 179.5 metres. The natural water contents in the upper crust ranged from 16 to 20 per cent.

The lower portion of the silty clay till in boreholes A and B had N values ranging from 4 to 63 blows per 0.3 metres. Standard penetration test N values in the previous boreholes ranged from 7 to 17 blows per 0.3 metres in the silty clay till. The natural water contents ranged from 12 to 36 per cent. In situ vane testing carried out in the softer portions of the silty clay till at and below elevation 179 metres indicated undrained shear strengths ranging from 34 to greater than 144 kilopascals. Sensitivities of 1.7 to 2.1 were determined based on the remoulded strengths.

The results of the laboratory consolidation testing carried out on sample 15 from borehole B are provided on Figures A-4 and A-5. The results indicate that the silty clay till is slightly preconsolidated by about 35 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_o	C_r	C_c	C_v cm²/sec
B-10	9.3	125	160	1.28	0.45	0.043	0.114	4×10^{-3}

The silty clay till encountered in the boreholes had average plastic and liquid limits of 14 and 33 per cent, respectively, indicating a clay soil of low plasticity. The results of the Atterberg limit determinations are shown on Figure A-1 in Appendix A.

Grain size distribution curves for the samples of the silty clay till recovered from the standard penetration testing are provided on Figure A-3.

4.1.4 Sand

Within the silty clay till in borehole B, a layer of dense fine to medium sand some 5.8 metres thick was encountered at elevation 155.7 metres. The sand had measured N values of 47 and 49 blows per 0.3 metres and natural water contents of 19 and 23 per cent.

4.1.5 Sandy Silt Till

Underlying the clayey silt till at borehole 75-3 was a layer of sandy silt till. Borehole 75-3 was terminated in this layer after exploring it for about 1.0 metre from elevation 172.2 metres. This deposit had an N value of 10 blows per 0.3 metres and a water content of 12 per cent.

4.1.6 Lower Clayey Silt Till

Beneath the sand in borehole B, and the silty clay till layer in borehole 75-3 layers of clayey silt till were encountered. The lower clayey silt till layer at borehole B had sand and gravel layers and boulders. The lower clayey silt till layers were some 4.0 and 3.1 metres thick and encountered at elevations 149.9 and 173.7 metres in boreholes B and 75-3 respectively.

The lower very stiff to hard clayey silt till layers had N values of 17 blows per 0.3 metres in borehole 75-3 and over 100 blows per 0.3 metres in borehole B. The lower clayey silt had natural water contents ranging from 7 to 11 per cent with an average water content of about 10 per cent.

A sample of the lower clayey silt till retrieved from borehole B had plastic and liquid limits of 12 and 22 per cent, respectively, indicating a clay soil of low plasticity. The results of the Atterberg limit determinations are shown on Figure A-1 in Appendix A.

A grain size distribution curve for a single clayey silt till sample recovered from the standard penetration testing is provided on Figure A-2.

4.1.7 Bedrock

Borehole B was terminated in bedrock after exploring it for some 60 millimetres by mud rotary tricone drilling techniques. The inferred surface of the bedrock is at elevation 145.9 metres.

4.2 Groundwater Conditions

Groundwater conditions were observed in the boreholes during drilling. All boreholes remained dry during drilling. Based on the change in water content and soil colour, the long-term groundwater level is expected to be between elevations 180 and 183 metres, within the brown silty clay till and above the grey silty clay till.

The measured Pike Creek water level was at elevation 182.94 metres on August 3, 2004. The 100 year high water level is 184.64 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 and Mr. Steve Loomis 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.

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PART B

FOUNDATION DESIGN REPORT
PIKE CREEK BRIDGE REPLACEMENT
HIGHWAY 401, SITE 6-075
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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 Pike Creek Bridge based on our interpretation of the factual information obtained during the investigation. These recommendations will be revisited and expanded when further detail is available during the preliminary design phase of the project. 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 Bridge within the project limits.

Based on the available information, it is understood that the existing single span, rigid frame bridge over Pike Creek will be demolished and replaced by a new 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 have an approximate bridge deck elevation of 187.0 metres. Design information for the existing structure indicates that the structure was erected in 1951 and is founded on three shallow strip footings near elevation 182.0 metres. The approximate elevation of the existing bridge deck is 186.5 metres. The bridge reconstruction 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 over extensive deposits of very stiff to hard becoming firm with depth clayey silt till and silty clay till. Compact sandy silt till was encountered in borehole 75-3 at elevation 172.2 metres. A 6 metre thick deposit of dense sand was intercepted at elevation 155.7 metres in borehole B which was underlain by some 4 metres of hard clayey silt till. Borehole B was terminated in bedrock which was encountered at elevation 145.9 metres. All boreholes were dry during drilling.

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 about elevation 183 metres on the very stiff to hard silty clay till or clayey silt till. Factored geotechnical resistance values of 425 kilopascals at Ultimate Limit States (ULS) and 275 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 due to the 0.5 metre grade raise required for the higher bridge deck. Differential settlement may also occur due to fill placement for the embankment widenings. However, modifications to 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 softened/disturbed during excavation and footing construction, the following angle of friction between the concrete and the founding soils, and corresponding coefficient of friction, tan δ, may be used:

Footings on silty clay till	angle of friction	28°
	tan δ	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 mud coat 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 mud coat. 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 mud coat be placed immediately after the footing is inspected.

6.4 Deep Foundations

End bearing piles driven to practical refusal in the hard clayey silt till at about elevation 148 metres are considered suitable for support of the abutments for the planned 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 or on the underlying bedrock may be taken as 1,600 kilonewtons (kN). The till and underlying bedrock are considered to be unyielding layers and, therefore the geotechnical resistance at SLS does not apply. The pile 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 widening of the approach embankments are 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 abutments. 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 (EPS), 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 kilonewtons 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:

Pile Spacing in Direction of Loading, d = Pile Diameter	*Subgrade Reaction Reduction Factor R*
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 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(l) 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(l).
- 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:

	GRANULAR A	GRANULAR B (Type III)
Soil unit weight:	22 kN/m³	21 kN/m³
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 0.6 metres due to the increased bridge deck height. The exact extent of the change in the embankment fills is not currently known. 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.

Based on our understanding of the amount of grade raise, embankment settlements are estimated to be 10 millimetres in the area of the core of the existing embankments and 20 millimetres in the embankment widening areas.

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. Pumping from well filtered sumps located at the base of the excavations may be required to provide groundwater control during foundation construction. 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 below the groundwater table at this site would be classified as Type 3 soils. The underlying cohesive deposits would be classified as Type 2.

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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

AS	auger sample
CS	chunk sample
DO	drive open
DS	Denison type sample
FS	foil sample
RC	rock core
ST	slotted tube
TO	thin-walled, open
TP	thin-walled, piston
WS	wash sample
SS	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.).

WH sampler advanced by static weight-weight, hammer

PH sampler advanced by hydraulic force

PM sampler advanced by manual force

III. SOIL DESCRIPTION

(a) *Cohesionless Soils*

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

(b) *Cohesive Soils*

Consistency	"Cu" = "Su" 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

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

$\pi = 3.1416$
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
υ	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*	in terms of total stress $\tau_f = cu+\sigma\tan\phi_u$
ϕ_u	apparent angle of shearing resistance, or friction	
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.

Golder Associates
London, Ontario

RECORD OF BOREHOLE No A

METRIC

PROJECT 041-130054-0-4
G.W.P. 62-00-00
DIST 1 HWY 401
DATUM GEODETIC

LOCATION N 4678012.7 ;E 272577.6
BOREHOLE TYPE POWER AUGER (HOLLOW STEM)
DATE August 23, 2004 - August 24, 2004

ORIGINATED BY MA
COMPILED BY BG
CHECKED BY DUP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.62	GROUND SURFACE							
0.09	TOPSOIL, clayey Brown							
	(FILL), silt with clay, some sand, trace gravel Loose Brown				185			
		1	SS	10			o	
		2	SS	8	184			
183.49 / 2.13	SILTY CLAY, trace to some sand, trace gravel (TILL) Very stiff to stiff Brown and grey fissured becoming brown below elev. 182.7m and grey at elev. 181.62m	3	SS	22	183		o	
		4	SS	20	182			
		5	SS	14			o	
		6	SS	9	181	>144+		
		7	SS	9	180 / 179		Wp —o— WL	
		8	SS	9	178	1.4; >144+	o	1 25 41 34
		9	SS	7	177 / 176	1.6; >144+	o	
		10	SS	7	175 / 174	1.5; 1.4		
172.97 / 12.65		11	SS	6	173		o	
	END OF BOREHOLE Borehole dry during drilling Aug 23 & 24, 2004							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No B

 METRIC

PROJECT 041-130054-0-4
G.W.P. 62-00-00
DIST 1 HWY 401
DATUM GEODETIC

LOCATION N 4678013.3 ;E 272558.3
BOREHOLE TYPE POWER AUGER (HOLLOW STEM) / TRI CONE MUD ROTARY
DATE July 29, 2004 - August 3, 2004

ORIGINATED BY SL
COMPILED BY BG
CHECKED BY DUP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.54	GROUND SURFACE					
0.00	TOPSOIL, clayey Brown					
0.23	CLAYEY SILT, trace sand, trace gravel (TILL) Very stiff to hard Brown				185	
		1	SS	22		
		2	SS	35	184	
		3	SS	27	183	
182.19		4	SS	21		
3.35	SILTY CLAY, trace to some sand, trace gravel (TILL) Firm to hard Grey				182	
		5	SS	12	181	
		6	SS	16		
		7	SS	10	180	
		8	SS	8	179	1 23 44 32
		9	SS	6	178	
		10	TO	PH	177	21.1 CONSOLIDATION
					176	
		11	SS	6	175	
					174	
		12	SS	5	173	
					172	
		13	SS	4	171	

Plot headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; STRAT PLOT; GROUND WATER CONDITIONS

Field vane sensitivity values plotted: 1.1, 2.1, 1.6, 1.1, 0.9, 0.7, 1.0, 1.1, 1.0, 0.8, 0.7, 0.8

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No B

METRIC

PROJECT	041-130054-0-4
G.W.P.	62-00-00
DIST	1
HWY	401
DATUM	GEODETIC
LOCATION	N 4678013.3 ;E 272558.3
BOREHOLE TYPE	POWER AUGER (HOLLOW STEM) / TRI CONE MUD ROTARY
DATE	July 29, 2004 - August 3, 2004
ORIGINATED BY	SL
COMPILED BY	BG
CHECKED BY	DUP

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
	SILTY CLAY, trace to some sand, trace gravel (TILL) Firm to hard Grey
155.67	

SAMPLES

NUMBER	TYPE	"N" VALUES	ELEVATION SCALE (approx.)	SHEAR STRENGTH kPa (sensitivity)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
14	TO	PH	170	+ 0.8, + 0.9	
15	SS	5	168.6	+ 1.0, + 0.8	
16	SS	3	167		4 10 45 42
17	SS	10	164		
18	SS	24	161		
19	SS	63	158		

Elevation scale: 170, 169, 168, 167, 166, 165, 164, 163, 162, 161, 160, 159, 158, 157, 156

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity   ○ 3% STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No B

 METRIC

PROJECT 041-130054-0-4
G.W.P. 62-00-00
DIST 1 HWY 401
DATUM GEODETIC

LOCATION N 4678013.3 ;E 272558.3
BOREHOLE TYPE POWER AUGER (HOLLOW STEM) / TRI CONE MUD ROTARY
DATE July 29, 2004 - August 3, 2004

ORIGINATED BY SL
COMPILED BY BG
CHECKED BY DUP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
29.87	SAND, fine to medium, trace silt Dense Grey						
		20	SS	49	155	o	
153.54 32.00	SAND, fine, some silt Dense Grey						
		21	SS	47	152	o	
149.88 35.66	CLAYEY SILT (TILL), with sand and gravel layers, with boulders Hard Grey						
		22	SS	108	149	o	10 44 35 12
		23	SS	100/ 75mm	147	o	
145.92 39.68	BEDROCK (Inferred) END OF BOREHOLE Borehole dry during drilling July 29,30 and Aug. 3, 2004	24	SS	100/ 50mm	146		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

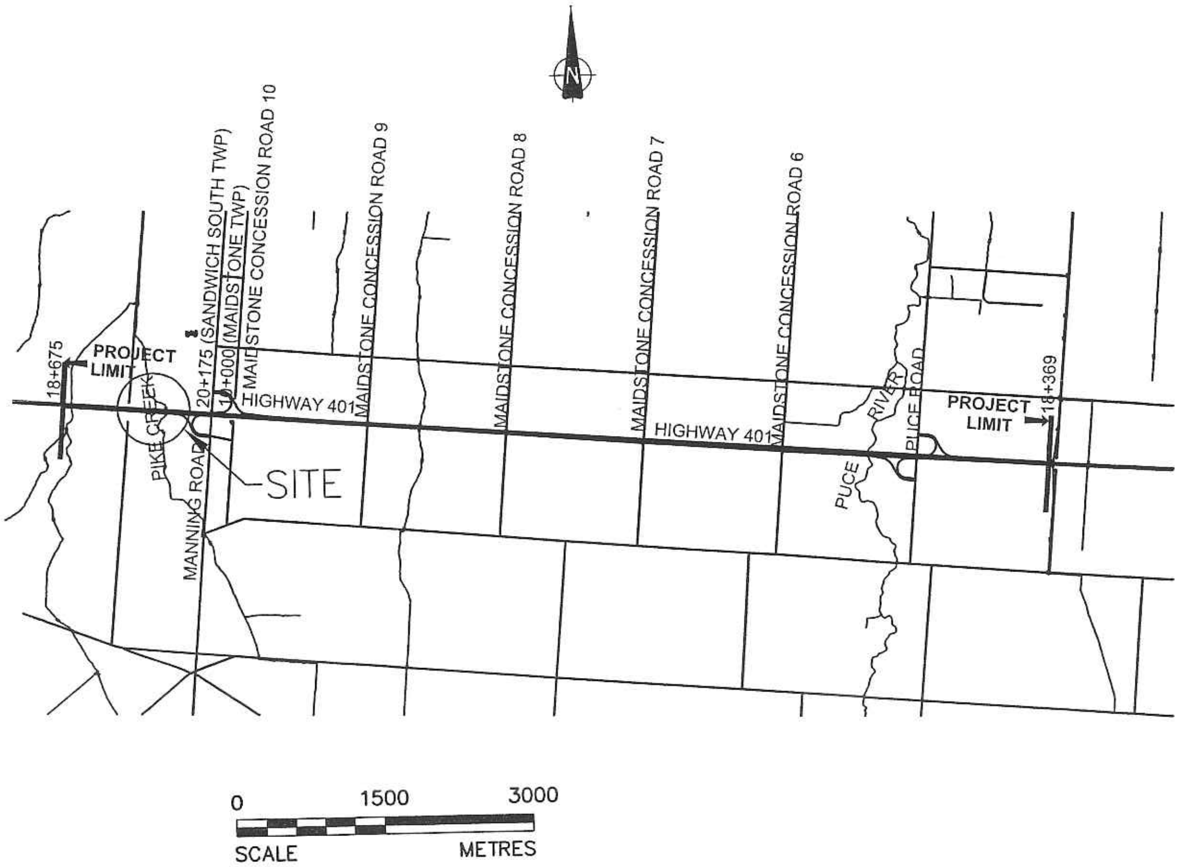
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

ON_MTO 041130054-0-4-PIKE.GPJ ON_MOT.GDT 12/24/04

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



PROJECT	PIKE CREEK BRIDGE G.W.P. 62-00-00 HIGHWAY 401		
TITLE	KEY PLAN		
Golder Associates LONDON, ONTARIO	PROJECT No. 041130005404		FILE No. 041-1300054-0-4F01 SCALE NTS REV. 0
	CADD JDR	DEC. 2004	FIGURE. 1
	CHECK DUP	DEC. 2004	

Drawing file: 041-1300054-0-4F01.dwg Dec 22, 2004 - 3:44pm

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

DIST HWY. 401
CONT. No. 2005–3001
WP No. 62–00–00

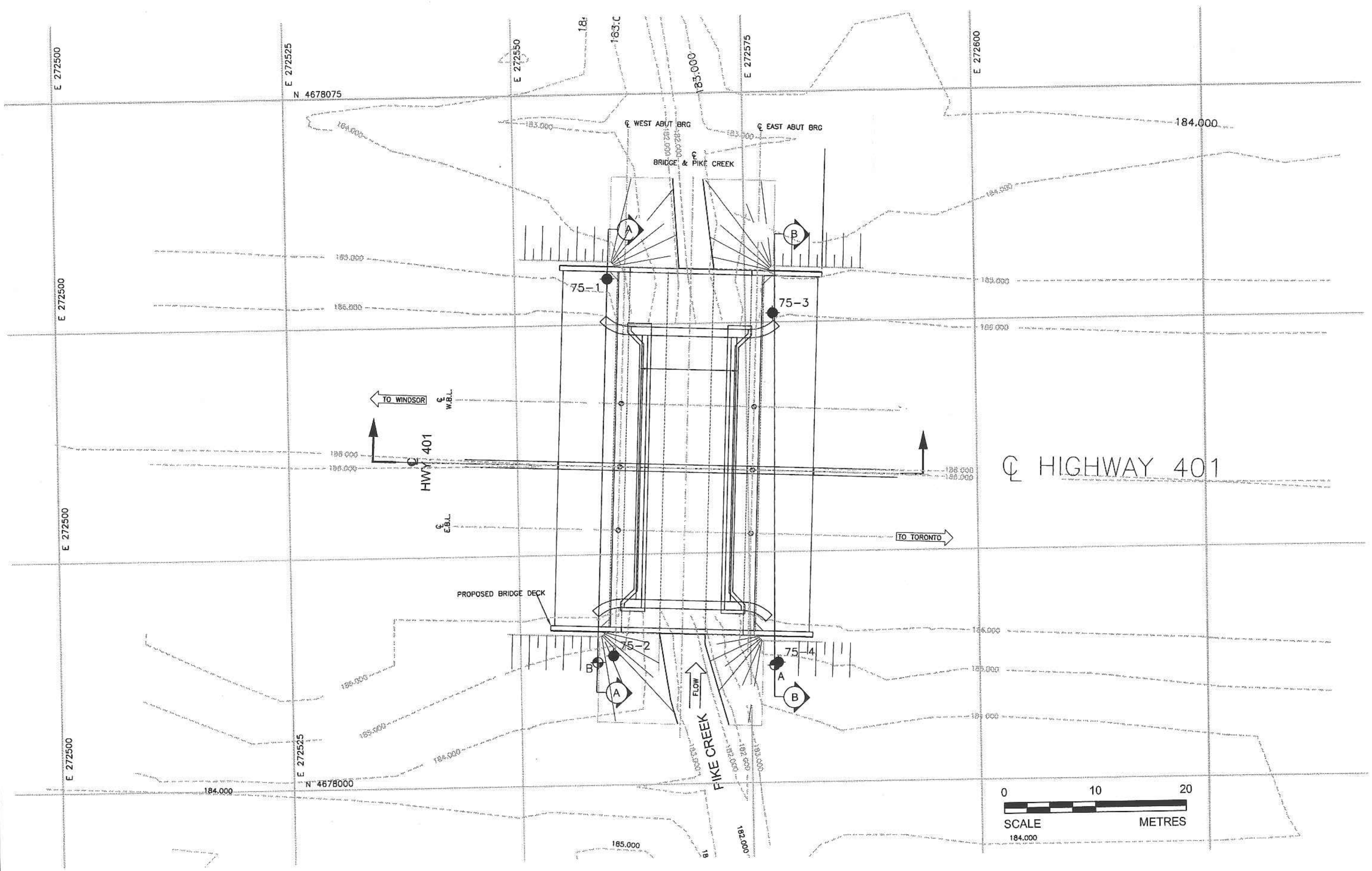
SHEET

PIKE CREEK BRIDGE

BOREHOLE LOCATIONS & SOIL STRATA

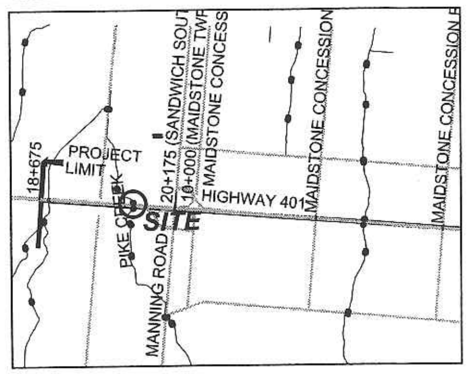
Golder Associates

Golder Associates Ltd.
LONDON, ONTARIO, CANADA



REFERENCE
DRAWING SUPPLIED BY DILLON CONSULTING, ENTITLED: PIKE RIVER BRIDGE

GENERAL ARRANGEMENT GWP - 62-00-00, FILE 'PIKE CREEK-GA_GOLDERS.dwg'



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	CO–ORDINATES EAST
75–1	184.8	4678055.0	272560.0
75–2	185.0	4678014.0	272560.0
75–3	185.4	4678051.0	272578.0
75–4	184.8	4678013.0	272578.0
A	185.6	4678012.7	272577.6
B	185.5	4678013.3	272558.3

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. 40J2–64

HWY. No. 401		PROJECT NO.: 041–130054–0–4	
SUBM'D. –	CHKD: –	DATE: DEC. 2004	
DRAWN: JDR	CHKD. DUP	APPD.	DWG. 1

00132250001.DWG

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

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

SHEET

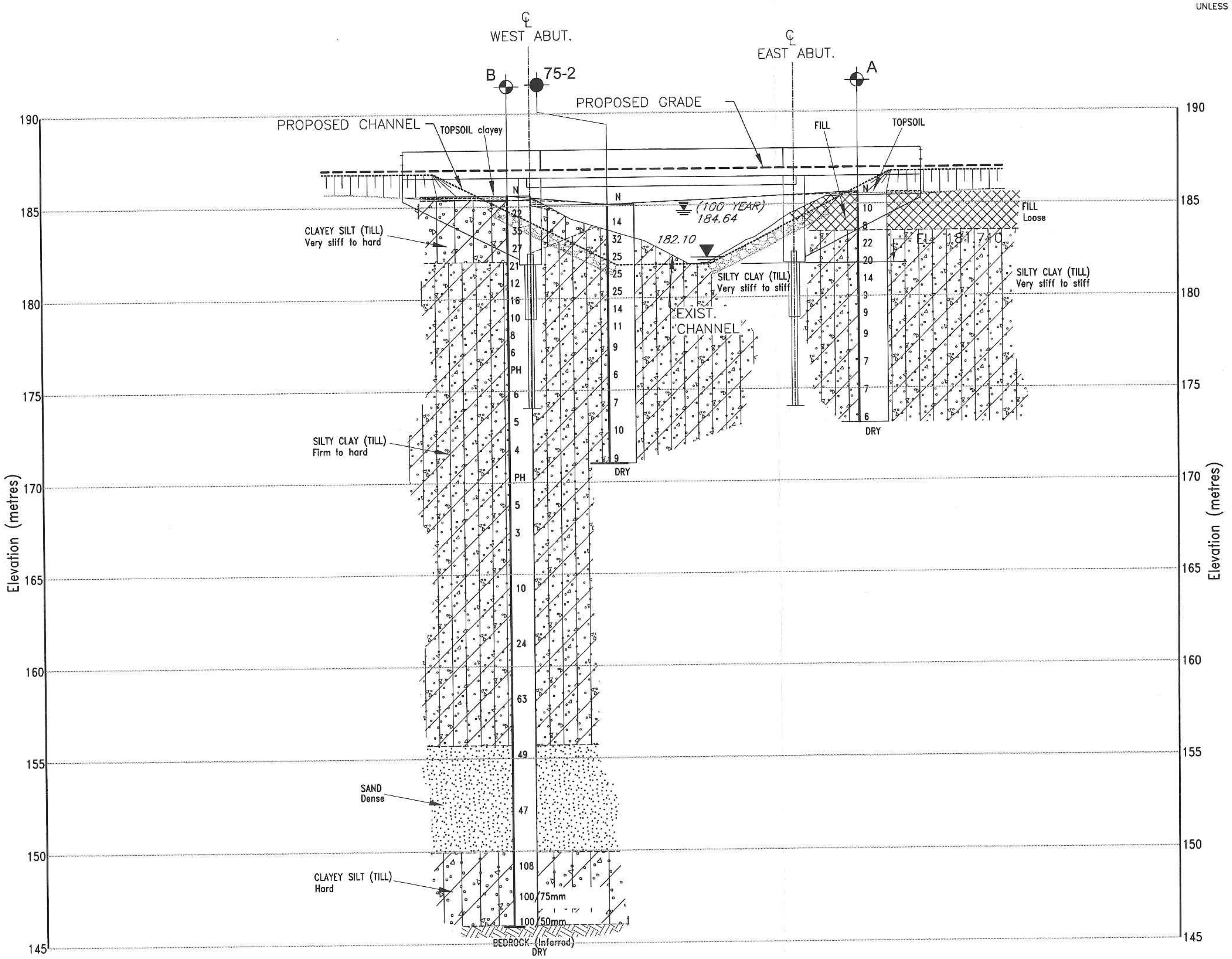
PIKE CREEK BRIDGE

BOREHOLE LOCATIONS & SOIL STRATA

Golder Associates
Golder Associates Ltd.
LONDON, ONTARIO, CANADA

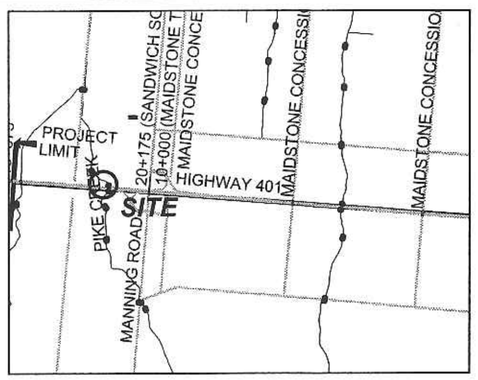
REFERENCE
DRAWING SUPPLIED BY DILLON CONSULTING, ENTITLED: PIKE RIVER BRIDGE

GENERAL ARRANGEMENT GWP - 62-00-00, FILE 'PIKE CREEK-GA_GOLDERS.dwg'



CENTRELINE PROFILE

0 5 10
SCALE METRES



KEY PLAN

LEGEND

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

No.	ELEVATION (metres)	CO-ORDINATES NORTH	CO-ORDINATES EAST
75-1	184.8	4678055.0	272560.0
75-2	185.0	4678014.0	272560.0
75-3	185.4	4678051.0	272578.0
75-4	184.8	4678013.0	272578.0
A	185.6	4678012.7	272577.6
B	185.5	4678013.3	272558.3

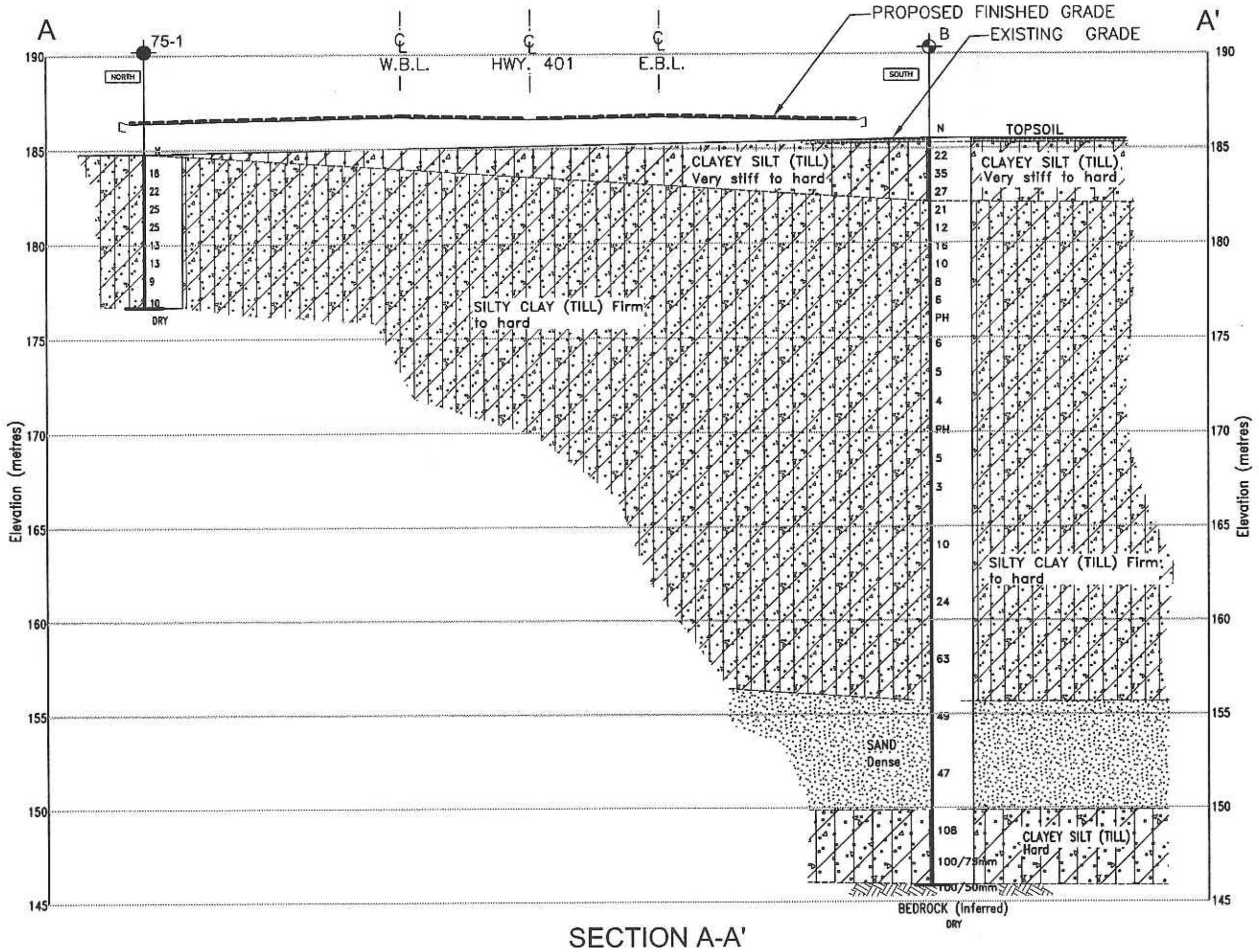
NOTES
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

NO.	DATE	BY	REVISION

Geocres No. 40J2-64

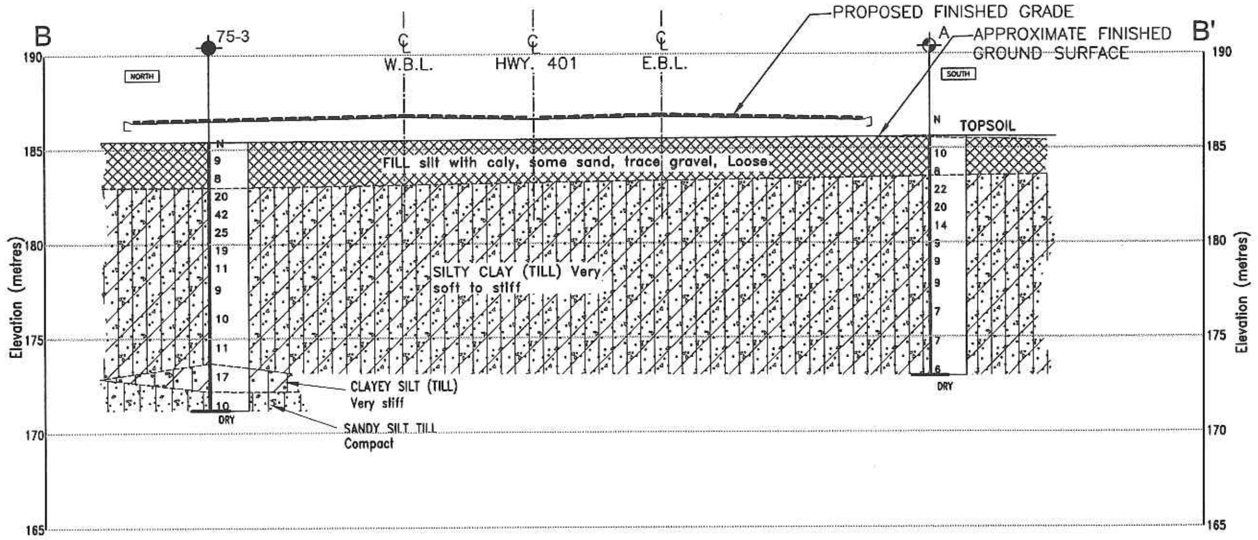
HWY. No. 401		PROJECT NO.: 041-130054-0-4	
SUBM'D. –	CHKD: –	DATE: DEC. 2004	
DRAWN: JDR	CHKD. DUP	APPD.	DWG. 2

001322SD001.DWG



SECTION A-A'

0 8 16
SCALE METRES



SECTION B-B'

0 8 16
SCALE METRES

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

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

PIKE CREEK BRIDGE

SOIL STRATA

SHEET

Golder Associates
Golder Associates Ltd.
LONDON, ONTARIO, CANADA

REFERENCE
DRAWING SUPPLIED BY DILLON CONSULTING, ENTITLED: PIKE RIVER BRIDGE

GENERAL ARRANGEMENT GWP - 62-00-00, FILE 'PIKE CREEK-GA_GOLDERS.dwg'

KEY PLAN

LEGEND

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

No.	ELEVATION (metres)	CO-ORDINATES NORTH	EAST
75-1	184.8	4678055.0	272560.0
75-2	185.0	4678014.0	272560.0
75-3	185.4	4678051.0	272578.0
75-4	184.8	4678013.0	272578.0
A	185.6	4678012.7	272577.6
B	185.5	4678013.3	272558.3

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

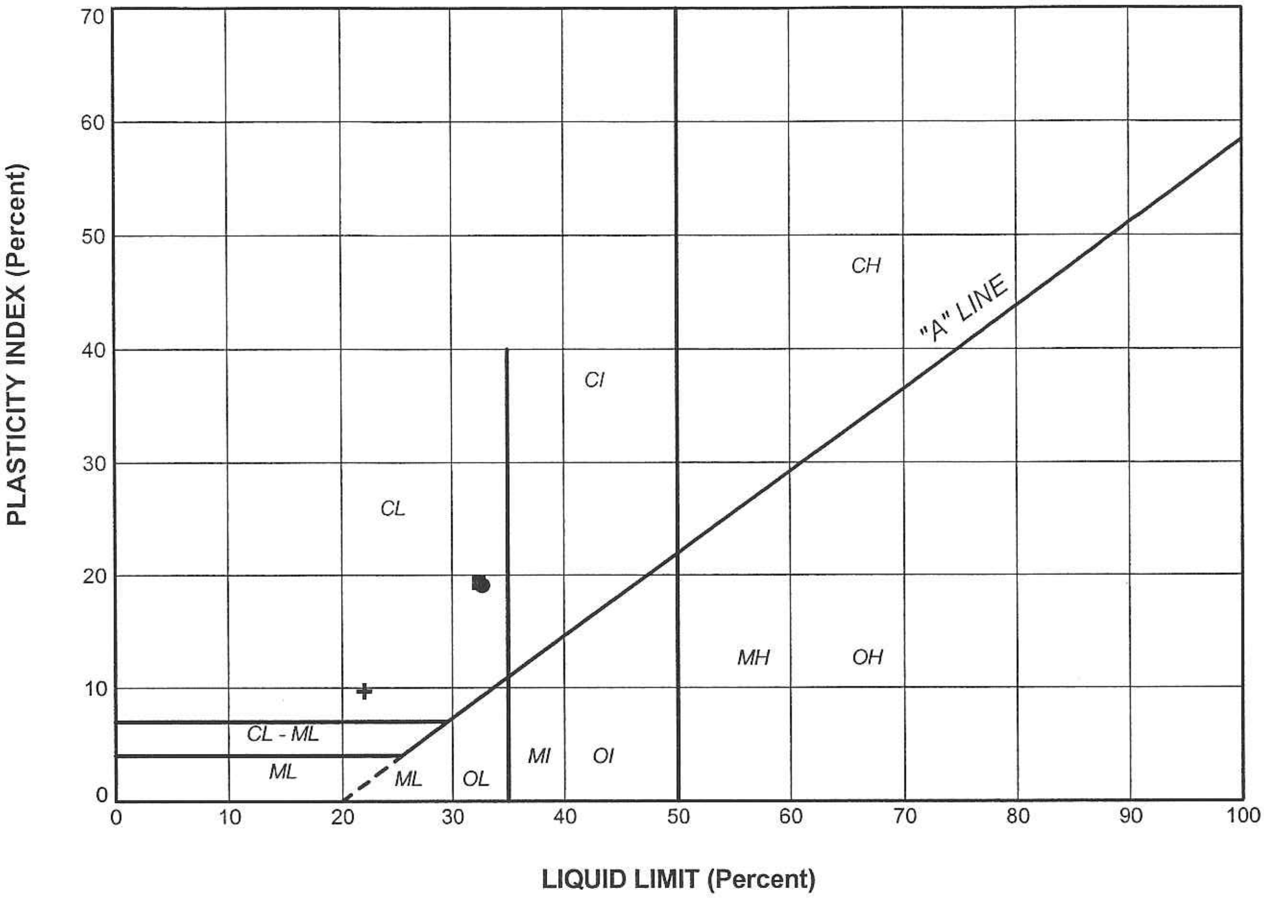
Geocres No. 40J2-64

HWY. No. 401		PROJECT NO.: 041-130054-0-4	
SUBM'D. -	CHKD: -	DATE: DEC. 2004	
DRAWN: JDR	CHKD. DUP	APPD.	DWG. 3

0013225D001.DWG

APPENDIX A

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



SOIL TYPE
C = Clay
M = Silt
O = Organic

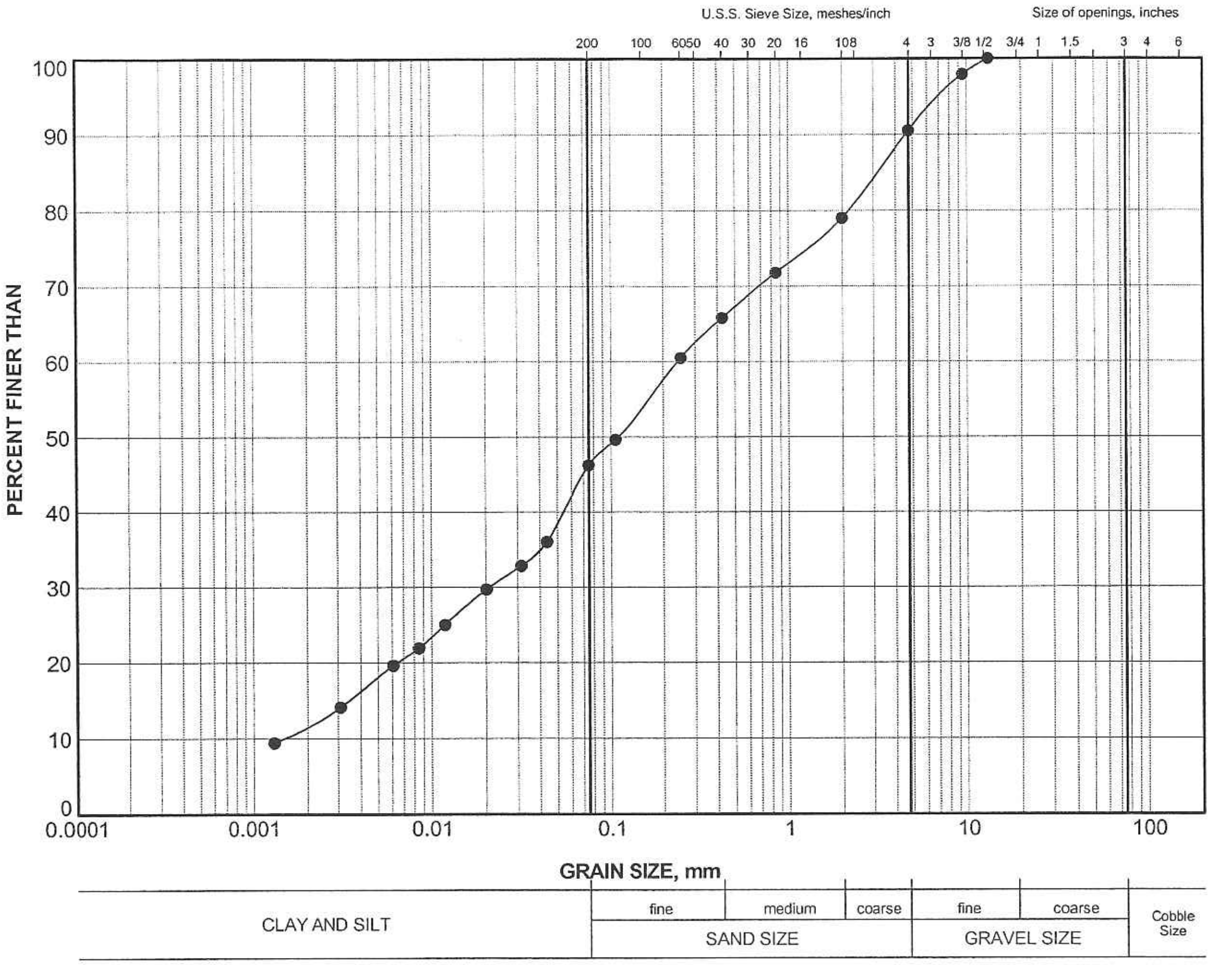
PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)	LL(%)	PL(%)	PI
●	A	7	179.5	32.7	13.6	19.1
■	B	9	177.9	32.4	13.0	19.4
▲	B	15	168.8	32.5	13.2	19.3
+	B	22	149.0	22.0	12.3	9.7

PROJECT	PIKE CREEK BRIDGE G.W.P. 62-00-00 HIGHWAY 401				
TITLE	**PLASTICITY CHART**				
Golder Associates LONDON, ONTARIO	PROJECT No.	041-130054-0-4		FILE No.	041130054-PIKE.GPJ
				SCALE N/A	REV.
	DRAWN	BG	Nov 12/04	**FIGURE A-1**	
	CHECK	DUP	Dec. 24/04		

MTO_PI GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	B	22	148.7

PROJECT

PIKE CREEK BRIDGE
G.W.P. 62-00-00
HIGHWAY 401

TITLE

GRAIN SIZE DISTRIBUTION

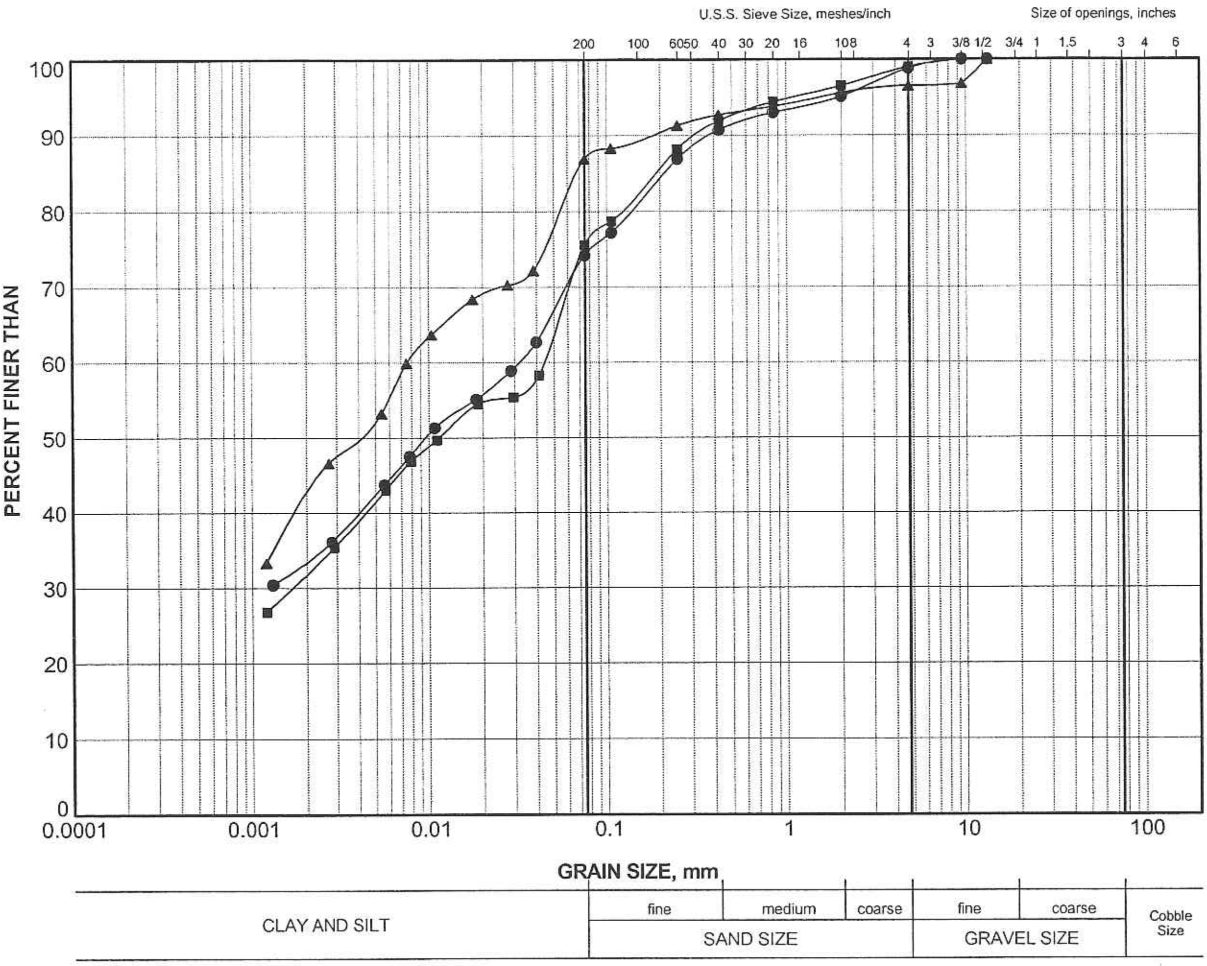
CLAYEY SILT TILL

Golder Associates
LONDON, ONTARIO

PROJECT No.	041-130054-0-4	FILE No.	041130054-PIKE.GPJ
		SCALE N/A	REV.
DRAWN	BG	Nov 12/04	
CHECK	DUP	Dec 24/04	

FIGURE A-2

LDN_MTO_NEW GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	A	8	177.8
■	B	8	179.1
▲	B	16	166.9

PROJECT

PIKE CREEK BRIDGE
G.W.P. 62-00-00
HIGHWAY 401

TITLE

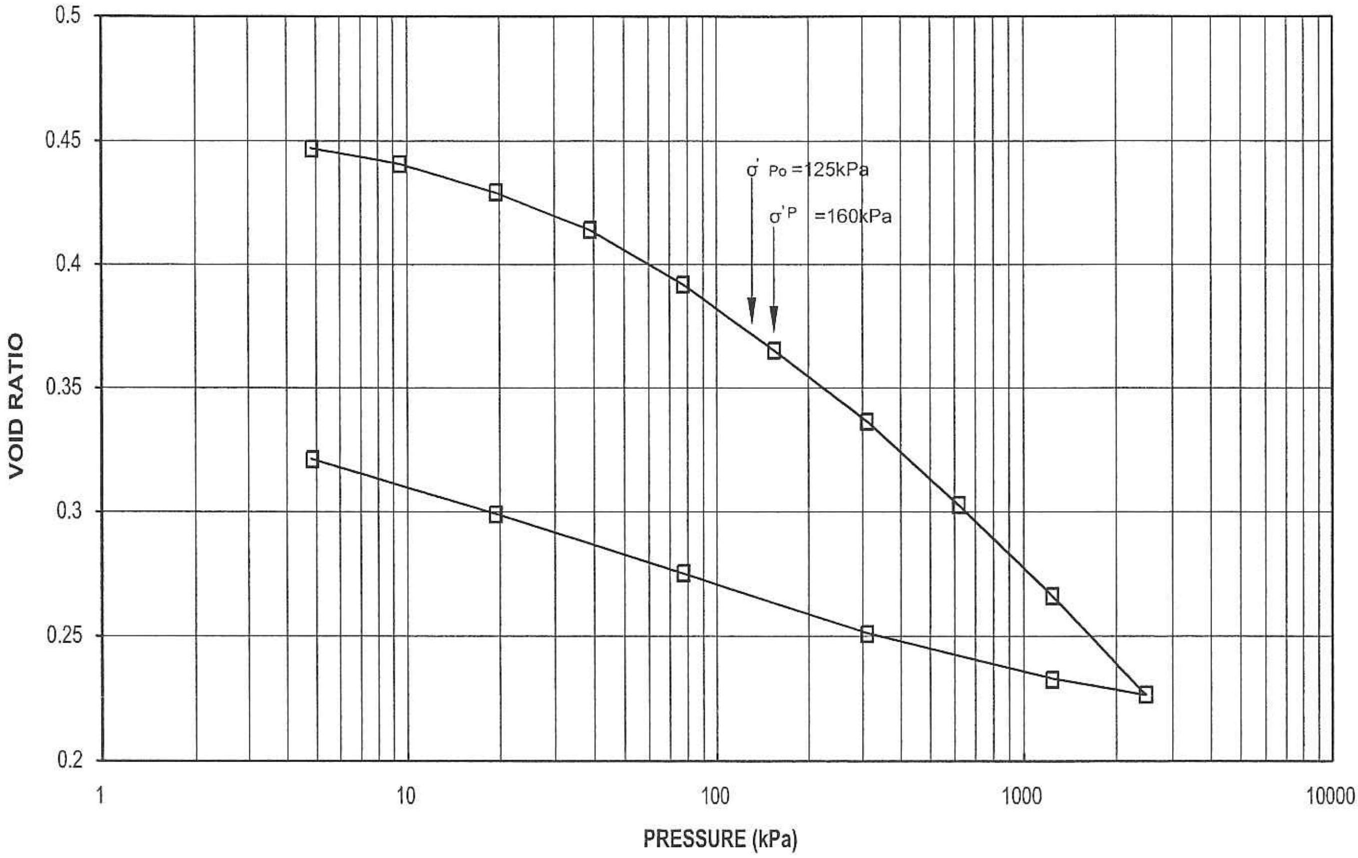
GRAIN SIZE DISTRIBUTION

SILTY CLAY TILL

Golder Associates
LONDON, ONTARIO

PROJECT No.	041-130054-0-4		FILE No.	041130054-PIKE.GPJ
			SCALE N/A	REV.
DRAWN	BG	Nov 12/04		
CHECK	DUP	Dec 24/04	FIGURE A-3	

LDN_MTO_NEW GLDR_LDN.GDT



BOREHOLE B, SAMPLE 10
ELEV. 176.2m

PROJECT

PIKE CREEK BRIDGE
GWP 62-00-00
HIGHWAY 401

TITLE

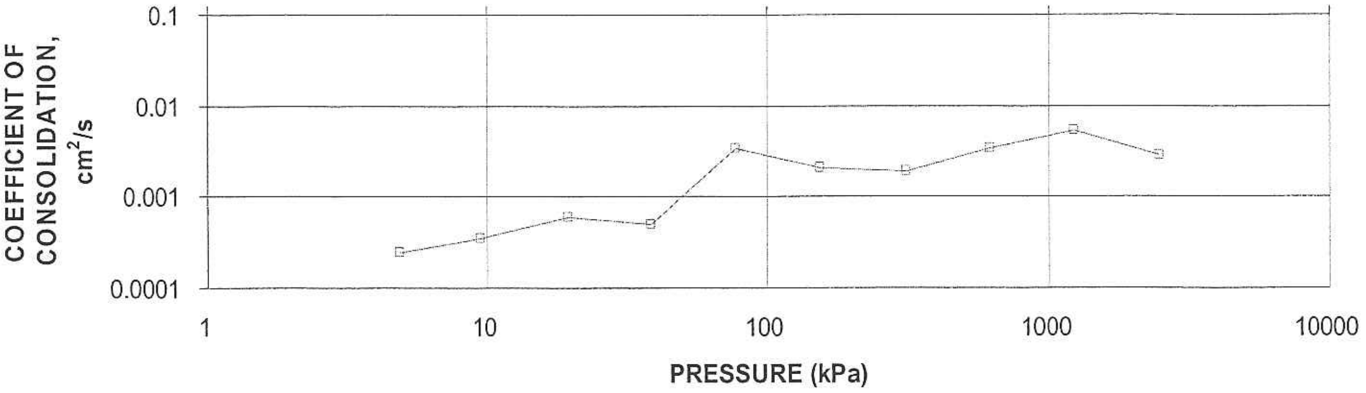
CONSOLIDATION TEST
VOID RATIO Vs. LOG PRESSURE

Golder Associates
LONDON, ONTARIO

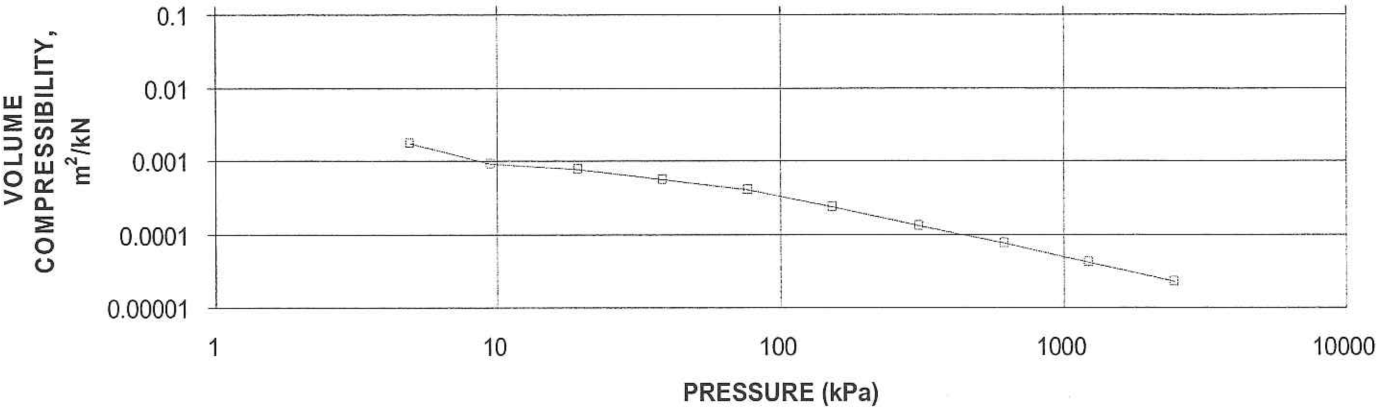
PROJECT No.		0411300540-4	FILE No.	041-130054-0-4
			SCALE AS SHOWN	REV. 0
CADD	BG	Nov. 12/04	FIGURE A-4	
CHECK	DP	Dec. 22/04		

Drawing file: figure A-4.dwg Dec 22, 2004 - 11:57am

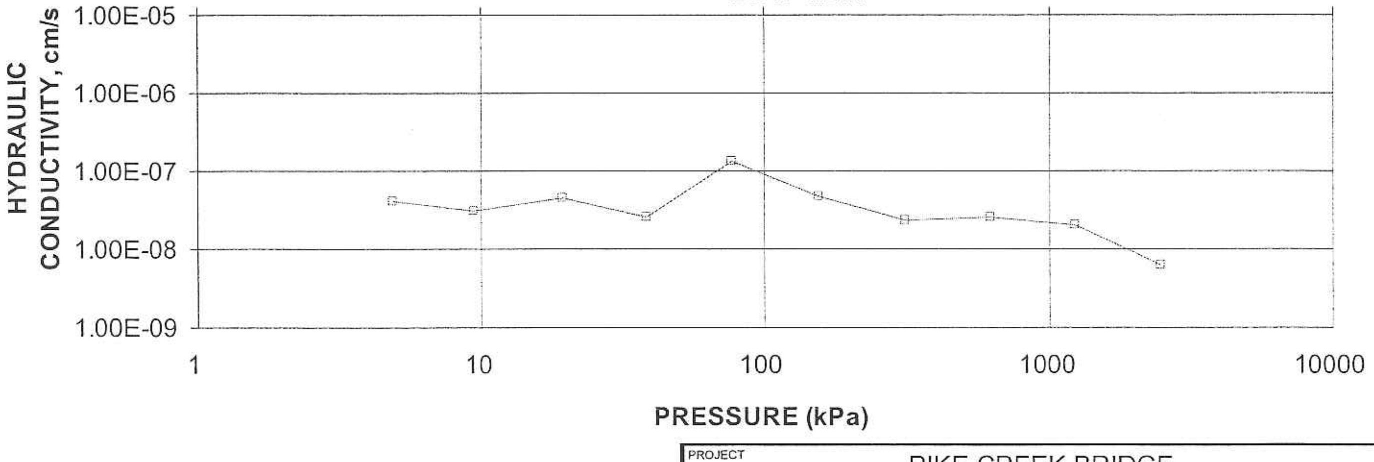
CONSOLIDATION TEST
CV cm^2/s VS PRESSURE (kPa)
BH B SA 10



CONSOLIDATION TEST
MV m^2/kN vs PRESSURE (kPa)
BH B SA 10



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH B SA 10



PROJECT
PIKE CREEK BRIDGE
GWP 62-00-00
HIGHWAY 401

TITLE
CONSOLIDATION TEST
VOID RATIO Vs. LOG PRESSURE

Golder Associates
LONDON, ONTARIO

PROJECT No.		0411300540-4	FILE No.	041-130054-0-4
			SCALE AS SHOWN	REV. 0
CADD	BG	Nov. 12/04	FIGURE A-5	
CHECK	DP	Dec. 22/04		

Drawing file: figure A-5.dwg Dec 22, 2004 - 12:12pm

APPENDIX B

RECORDS OF BOREHOLES BY OTHERS

Ontario

Peto MacCallum Ltd.

Foundation Design

RECORD OF BOREHOLE No 75-1

METRIC

G.W.P. [illegible]	LOCATION [illegible]	ORIGINATED BY [illegible]	
DIST 32	HWY 401	BOREHOLE TYPE [illegible]	COMPILED BY [illegible]
DATUM Geodetic	DATE [illegible]	CHECKED BY [illegible]	

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
184.80	Ground Level						
0.00	Silty clay, with sand, trace of gravel Very Stiff Brown (Till)		1	SS	16		184
			2	SS	22		183
			3	SS	25		182
			4	SS	25		181
			5	SS	13		
	Stiff Gray		6	SS	13		180
							179
			7	SS	9		178
176.70			8	SS	10		177
8.10	End of Borehole Borehole dry on completion of drilling						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; X LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_p — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

$+^3, \times^5$ Numbers refer to Sensitivity

15 —◇— 5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 75-2

1 of 1 METRIC

C.W.P. [illegible] LOCATION [illegible] ORIGINATED BY [illegible]

DIST 3? HWY [illegible] BOREHOLE TYPE [illegible] COMPILED BY [illegible]

DATUM Geodetic DATE [illegible] CHECKED BY [illegible]

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
195.03	Ground Level				
0.00	Silty clay, trace of sand, Stiff, Brown Mottled (Fill)				
194.03 / 1.00	Silty clay, with sand, trace of gravel, Very Stiff to Hard, Brown (Till)	1	SS	14	
		2	SS	32	
		3	SS	26	
		4	SS	2?	
		5	SS	26	
	Stiff to Firm, Grey	6	SS	14	1 22 15 ??
		7	SS	11	
		8	SS	9	
		9	SS	8	6 21 37 36
		10	SS	7	
		11	SS	10	
170.83 / 14.20	End of Borehole. Borehole dry on completion of drilling. ■ Penetrometer Test	12	SS	9	

Elevation scale: 194, 193, 192, 191, 190, 179, 178, 177, 176, 175, 174, 173, 172, 171

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity; 15 -○- 5 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 7S-3

 METRIC

G.W.P. 60-00-00 LOCATION Co-ords. [illegible] ORIGINATED BY [illegible]
DIST 12 HWY 401 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY [illegible]
DATUM Geodetic DATE May 14, 200[illegible] CHECKED BY [illegible]

SOIL PROFILE

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
185.40	Ground Level			
0.00	Silty clay, trace to some sand, with org. inclusions of topsoil. Stiff. Brown (Fill)	1	SS	2
		2	SS	8
182.95 / 2.45	Silty clay, with sand, trace of gravel. Very Stiff to Hard. Brown (Till)	3	SS	20
		4	SS	52
	Grey	5	SS	25
		6	SS	19
	Stiff	7	SS	11
		8	SS	9
			FV	
		9	SS	10
			FV	
		10	SS	11
173.70 / 11.70	Clayey silt, some sand, trace of gravel. Very Stiff. Grey (Till)	11	SS	17
172.20 / 13.20	Sandy silt, trace of clay and gravel. Compact. Grey (Till). Wet	12	SS	10
171.20 / 14.20	End of Borehole. Borehole dry on completion of drilling. ■ Penetrometer Test			

GROUND WATER CONDITIONS

ELEVATION SCALE: 185, 184, 183, 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

Field vane values: +1.7, +1.9

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ON_MOT 01FF073B.GPJ ON_MOT.GDT 11/19/2002 11:50:35 AM

$+^{3}$, $\times^{3}$: Numbers refer to Sensitivity   15—○—3 (%) STRAIN AT FAILURE

Ontario [illegible] MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 75-4

1 of 1 METRIC

G.W.P. 60-00-00 LOCATION [illegible] ORIGINATED BY MP

DIST 33 HWY 401 BOREHOLE TYPE [illegible] Flight Solid Stem Augers COMPILED BY GD

DATUM Geodetic DATE May 07, 2002 CHECKED BY MRA

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100)	PLASTIC LIMIT w_p / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.85	Ground Level									
0.00	Silty clay, trace of sand, Stiff, Brown (FILL)	1	SS	9		184				
183.45 / 183.40 / 1.60	Silty clay, Dark Brown (Topsoil)	2	SS	[illegible]		183				
	Silty clay, with sand, trace of gravel, Very Stiff to Hard, Brown (TILL)	3	SS	34		182				
		4	SS	20						
	Very Stiff to Stiff, Grey	5	SS	16		181				
		6	SS	12		180				
						179				
178.30		7	SS	11						
6.55	End of Borehole. Borehole dry on completion of drilling. ● Penetrometer Test									

$+^{7}$, $\times^{3}$: Numbers refer to Sensitivity; 20 / 15 –○– 5 / 10 (%) STRAIN AT FAILURE

APPENDIX C

SITE PHOTOGRAPHS

SITE PHOTOGRAPHS

Photo 1: South Elevation

Photo 2: Bridge Deck Looking North