

**PRELIMINARY FOUNDATION INVESTIGATION
AND DESIGN REPORT
PROPOSED COLBORNE ROAD OVERPASS
STRUCTURE WIDENING
HIGHWAY 402, GWP 3038-03-00
AGREEMENT NUMBER 3005-A-000394**

Submitted to:

URS Canada Inc.
75 Commerce Valley Drive East
Markham, Ontario
L3T 7N9

DISTRIBUTION:

6 Copies - URS Canada Inc.
2 Copy - Golder Associates Ltd.

July 28, 2006

041-130099-7
Geocres No. 40J16-68

TABLE OF CONTENTS

<u>SECTION</u>	<u>PAGE</u>
PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT	
1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	2
3.0 INVESTIGATION PROCEDURES.....	3
4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY.....	4
4.1 Geology	4
4.2 Site Stratigraphy.....	5
4.2.1 Surficial Fill and Topsoil.....	5
4.2.2 Sand and Gravel.....	6
4.2.3 Sand, Silty Fine Sand and Sandy Silt	6
4.2.4 Peat and Organic Silt.....	6
4.2.5 Clayey Silt	7
4.2.6 Silty Clay	8
4.2.7 Bedrock	8
4.3 Groundwater Conditions.....	8
5.0 MISCELLANEOUS	10
PART B - PRELIMINARY FOUNDATION DESIGN REPORT	
6.0 ENGINEERING RECOMMENDATIONS.....	11
6.1 General	11
6.2 Bridge Widening Foundations	11
6.3 Shallow Foundations	12
6.4 Deep Foundations.....	12
6.4.1 Geotechnical Axial Resistance – Driven Steel Piles.....	13
6.4.2 Dondrag Load (Negative Skin Friction).....	13
6.4.3 Resistance to Lateral Loads	14
6.4.4 Frost Protection	15
6.5 Lateral Earth Pressures.....	15
6.6 Embankments	16
6.7 Excavations and Temporary Cut Slopes.....	17
7.0 MISCELLANEOUS	18

TABLE OF CONTENTS CONTINUED

In Order
Following
Page 18

TABLE I - Comparison of Foundation Alternatives

LIST OF ABBREVIATIONS

LIST OF SYMBOLS

RECORDS OF BOREHOLES

FIGURE 1 - Site Location Plan

DRAWING 1 - Borehole Locations and Strata Profile

DRAWING 2 –Stratigraphic Sections at Abutments

APPENDIX A – Laboratory Test Results

APPENDIX B - Records of Previous Boreholes

APPENDIX C – Site Photographs

PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
PROPOSED COLBORNE ROAD OVERPASS
STRUCTURE WIDENING
HIGHWAY 402, GWP 3038-03-00
AGREEMENT NUMBER 3005-A-000394**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations at various sites along Highway 402 in conjunction with GWP 3038-03-00 which extends from the Bluewater Bridge Authority plaza east for 16 kilometres to Lambton Road 26 (Mandaumin Road) in Sarnia, Ontario. This report addresses the widening of the existing overpass structure at Colborne Road to provide an additional two lanes for Highway 402.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed widening by utilizing new and existing borehole data. The terms of reference for the scope of work are outlined in Golder's Total Project Management (TPM) proposal P31-3109, dated December 2003 and amended by our letter dated June 22, 2005. The work was carried out in accordance with our Quality Control of TPM Services Plan, Agreement No. 3005-A-000394, dated May 2004.

URS provided Golder with a general arrangement drawing for the planned overpass structure widening on August 30, 2005. The widening will occur on the north side of the existing structure. According to the design drawings, the proposed works will include construction of new semi-integral abutments for the widening on the north side with conversion of the existing abutments to semi-integral abutments. The bridge abutments will be supported on piles.

2.0 SITE DESCRIPTION

The project area covered by this report is located on Highway 402, approximately 2.5 kilometres east of the east end of the Blue Water Bridge over the St. Clair River, at Colborne Road in Sarnia, Ontario. The subject site is located in a mixed use residential and commercial area of Sarnia. The existing structure is bounded by low and medium rise apartment blocks to the northwest, northeast and southwest of Colborne Road and Highway 402, Capel Street to the southwest, Guthrie Street on the northeast and the Northgate Shopping Centre on the southeast. The abutment slopes are currently well vegetated. The location of the site is shown on Figure 1.

The existing structure is a steel girder bridge with a 18.3 metre span which was erected in 1957 and rehabilitated in 1997. The bridge currently accommodates two lanes each of west bound and east bound Highway 402 traffic. General arrangement drawings for the existing bridge indicate that the abutments are founded on creosoted timber Class B piles which are 9.1 to 12.2 metres long. According to ASTM D 25-37 to 58, Class B timber piles under 12.2 metres in length have a minimum tip diameter of 203 millimetres and a diameter 0.91 metres from the butt which is in the range of 305 to 508 millimetres. The drawings indicate that, at each abutment, 214 piles with a maximum load of 20 tons per pile, were driven. The top of the pile caps is approximately at 182.6 metres. It is believed that Photo 1 in Appendix C illustrates the pile installation process at the Colborne Road site.¹

The approximate elevation of Colborne Road at this location is 183.4 metres. The approximate elevation of the bridge deck is 189.4 metres. The general ground surface elevation in the vicinity of the widening is about 183 metres.

¹ Pressure Treated Timber Piles: A Manual For Architects and Engineers (1st ed.). (1962). Photograph (pp.43). Ottawa, ON: Canadian Institute of Timber Construction.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between July 22 and 29, 2005. Two boreholes, C-1 and C-2, were drilled at the northeast and southwest corners of the structure, respectively. The boreholes were advanced to depths of 41.8 metres at C-1 and 13.4 metres at C-2. The borehole locations are shown in plan on Drawing 1.

The investigation was carried out using a track mounted CME 75 drill rig and a truck mounted CME 45 drill rig. Borehole C-1 was advanced using continuous flight hollow stem augers. Borehole C-2 was advanced using mud rotary drilling techniques with an N-sized tricone bit. In the boreholes, samples of the overburden were obtained at suitable intervals of depth using 50 millimetre outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedures. Groundwater conditions in the boreholes were observed throughout the drilling operations. A standpipe was installed in borehole C-2 in order to monitor the prevailing groundwater level. Both of the boreholes were backfilled using MTO recommended procedures and as required by Ontario Regulation 903 (amended by Ontario Regulation 128/03).

The field work was supervised on a full-time basis by members of our engineering staff who located the boreholes in the field, obtained utility locates, directed the drilling, sampling and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labeled containers and transported to our laboratory in London, Ontario for further examination. Index and classification tests, consisting of grain size analyses, water content and Atterberg Limits determinations were carried out on selected samples. The results of the field and laboratory testing are given on the Record of Borehole sheets and in Appendix A.

The as-drilled borehole locations and elevations were surveyed by J.D. Barnes Limited. The elevations at the boreholes are referenced to geodetic datum.

This report also incorporates information from a previous investigation which was carried out for the existing Colborne Road Overpass structure. The results of that investigation are contained within Department of Highways Ontario report 55-F-212C entitled "Soil Investigations for the Colborne Road Overpass, Sarnia, Ontario" dated April 4, 1955. At that time, four boreholes were put down at the site of the existing structure. Boreholes 1 to 4 were drilled and sampled to depths of 12.2 to 29.0 metres during the period March 10 to 18, 1955. The borehole locations are shown on the 'as-constructed' Department of Highways, Ontario Drawing No. TWP#179-38-1-A dated June 26, 1968.

The borehole locations are shown in plan on Drawing 1. The subsurface conditions encountered in the boreholes in the vicinity of the proposed widening are shown in profile on Drawings 1 and 2 in metric units. The results of the boreholes drilled for the existing Colborne Road Overpass structures are provided in Appendix B in their original imperial units and format. Corresponding depths and elevations in metric units been added to these Records of Boreholes.

The borehole locations and ground surface elevations are summarized as follows:

BOREHOLE NUMBER	BOREHOLE LOCATION		GROUND SURFACE ELEVATION (m)
	NORTHING (m)	EASTING (m)	
1	4760883.6	313862.1	183.00
2	4760882.9	313883.4	182.90
3	4760847.1	313860.8	183.30
4	4760846.4	313882.1	183.20
C-1	4760839.1	313858.0	183.03
C-2	4760896.0	313885.2	182.77

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Geology

The area of the site is located in the physiographic region known as the Huron Fringe². Geological information indicates that the general soil conditions for the area consist of glacial lacustrine deposits overlying deep lacustrine till deposits.

The surficial glaciolacustrine deposits represent the shoreline and near shores of former Lakes Algonquin and Nipissing. These deposits consist of sands, silts and minor amounts of gravel. The lacustrine tills underlying the surficial granular deposits and discontinuous layer of organic soils are referred to as the St. Joseph Tills and generally consist of silty clay to clayey silt materials deposited in glacial Lake Whittlesey or Lake Warren during the Wisconsin period of glaciation. The upper 3 to 5 metres of the till deposit has been desiccated and oxidized forming a crust, the lower extent of which corresponds to the long-term groundwater level in the deposit. The St. Joseph Tills are commonly separated from the underlying black shale bedrock by massive to laminated lacustrine sandy silt to clay.

The average overburden thickness is 34 metres and generally varies from about 30 to 40 metres in the area of the site. A previous study of bedrock in the area indicated that the elevation of the bedrock surface in the vicinity of the site varies between approximately 130 to 140 metres. The bedrock belongs to the Kettle Point Formation. It is black bituminous shale with greenish grey silty-shale interbeds. Beneath the Kettle Point Formation, the bedrock reportedly consists of a sequence of shale, limestone and dolomite of the Hamilton and Port Lambton Groups.

² L.J. Chapman and D.F. Putnam: The Physiography of Southern Ontario, Third Edition. Ontario Geological Survey, Special Volume 2, 1984.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes together with the results of the field and laboratory testing are shown on the attached Record of Borehole sheets and in Appendices A and B. The stratigraphic boundaries shown on the borehole sheets are inferred from non-continuous sampling and, therefore, may represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsurface conditions encountered at the site consist of topsoil, fill and compact surficial gravel, sand and gravel and sands exceeding 2 metres below the ground surface. About 3 to 5 metres of compact silty fine sand, fine sand and sandy silt underlie the surficial deposits. These granular layers were underlain by peat interlayered with generally very soft to firm clayey silt or organic silt or compact silty sand. The thickness of the zone with organic and soft cohesive materials was up to 5 metres thick on the north side of the structure and diminished to less than 0.5 metres on the south side. Beneath the organic deposits, a 2 to 5 metre thick layer of stiff clayey silt was encountered. The clayey silt was underlain by a layer of stiff silty clay. In the single borehole which was advanced into the underlying shale bedrock, the silty clay was found to be approximately 26 metres thick. The bedrock was encountered near elevation 143 metres, about 40 metres below the existing ground surface.

Locations and elevations of the boreholes are shown on the attached Drawing 1 along with the interpreted stratigraphic profile along the centerline of the proposed widening. Additional soil profiles are shown in Drawing 2. A detailed description of the subsurface conditions encountered in the boreholes for this investigation is provided on the Record of Borehole sheets and is summarized in the following sections.

4.2.1 Surficial Fill and Topsoil

Fill and topsoil materials were encountered only in the two boreholes advanced for the current investigation. This fill was likely placed during construction of the bridge embankments and associated roadworks. Fill was present from elevation 182.9 metres beneath surficial topsoil at borehole C-1 and from the ground surface at borehole C-2. The fill layers consisted of loose silty fine sand and fine sand which were 0.7 and 1.5 metres thick at the borehole locations. A single standard penetration test conducted in the fill had an N value of 6 blows per 0.3 metres. The fill samples had in situ water contents of 1 and 6 per cent.

Surficial topsoil 180 millimetres thick was encountered at borehole C-1. A 310 millimetre thick layer of buried topsoil was encountered beneath the fill in borehole C-2 at elevation 181.3 metres.

4.2.2 Sand and Gravel

Layers of sand and gravel or gravel were encountered in all four boreholes advanced for the 1955 investigation. Sand and gravel or gravel layers were present at the ground surface in boreholes 1, 2 and 3 and beneath the surficial sand layer at borehole 2 from elevation 182.3 metres. The sand and gravel layers were 0.9 to 2.4 metres thick with an average thickness of 1.9 metres.

The sand and gravel layers had N values of 5 to 35 blows per 0.3 metres with an average of 20 blows per 0.3 metres.

4.2.3 Sand, Silty Fine Sand and Sandy Silt

Sand deposits were found in all boreholes for the current and previous investigations. Silty fine sand was encountered beneath the sand and gravel layers in boreholes 1, 2, 3, 4 near elevation 181.0 metres. Lower silty fine sand layers were intercepted at elevation 176.9 metres between the peat layers in borehole 1 and at elevation 177.9 metres beneath the sand layer in borehole C-1. Fine sand was encountered at the surface of borehole 4, beneath the fill at elevation 182.1 metres in borehole C-1 and beneath the topsoil at elevation 181.3 metres in borehole C-2. Sandy silt was encountered at elevation 176.8 metres beneath the fine sand in borehole C-2.

The silty fine sand and fine sand layers were compact to dense and had recorded N values ranging from 10 to 35 blows per 0.3 metres with an average of 21 blows per 0.3 metres. The natural water content of the sand and silty fine sand was between 21 and 27 per cent with an average of 25 per cent. The sandy silt had an N value of 19 blows per 0.3 metres and a water content of 18 per cent.

Figure A-1 in Appendix A provides the gradation curves for two samples retrieved from the fine sand deposits.

4.2.4 Peat and Organic Silt

Layers of organic materials were noted in all boreholes except for borehole C-1 of the current investigation. Where present, the thickness of the organic layers ranged from 0.5 to 4.9 metres with the layer thicknesses increasing to the north and west.

At elevation 176.1 metres in borehole C-2, a 1.5 metre thick layer of soft clayey silt containing shells was encountered above a 1.5 metre thick deposit of layered peat and organic silt. Decayed vegetation, interpreted as peat, was encountered in boreholes 1 to 4 of the 1955 investigation. Deposits of loose to compact silty fine sand or very soft to hard clayey silt interlayered with peat were intercepted at elevation 178.1 metres in borehole 1, 175.6 metres in borehole 2, 177.5 metres in borehole 3 and 177.9 metres in borehole 4.

Standard penetration N values of 3 to 33 blows per 0.3 metres were recorded in the zone containing organic materials. The water contents of two samples of the organic soils from borehole C-2 were 76 and 251 per cent, respectively.

4.2.5 Clayey Silt

Clayey silt was encountered in both boreholes advanced for the current investigation. Boreholes from the previous investigation encountered several deposits described as clayey silt, silty clay and clay. For the purposes of this report, soils described in boreholes 1 to 4 as clayey silt or silty clay and described in borehole 4 as clay not containing 'pieces' of sand or fine gravel have been interpreted as clayey silt.

An upper layer of clayey silt was encountered beneath the silty fine sand and sandy silt deposits at elevation 176.3 metres in borehole 1, 175.6 metres in borehole 2, 177.5 metres in borehole 3, 177.9 metres in borehole 4, 177.4 metres in borehole C-1 and 176.1 metres in borehole C-2. With the exception of the 0.7 metre thick clayey silt layer at borehole C-1 which was interlayered with sand seams, the upper layer of clayey silt contained peat or shells. The upper clayey silt had standard penetration test N values ranging from the weight of the sampling hammer to 33 blows per 0.3 metres but generally less than 8 blows per 0.3 metres.

The upper layer of clayey silt and organic deposits was found to be underlain by a lower clayey silt layer at elevation 172.6 metres in borehole 1, 171.4 metres in borehole 2, 175.7 metres in borehole 3, 175.7 metres in borehole 3, 177.4 metres in borehole 4, 176.7 metres in borehole C-1 and 173.0 metres in borehole C-2. The clayey silt was found to have a very stiff to hard crust above approximate elevations 171 metres on the north and 173 metres on the south side of the existing structure. The clayey silt had standard penetration test N values between 4 and 45 blows per 0.3 metres with an average of 20 blows per 0.3 metres. The shear strength of the softer zone in the stratum was measured by in situ vane testing. In situ vane testing indicated undrained shear strengths ranging from 65 to greater than 144 kilopascals (kPa) indicating a stiff to very stiff consistency. Sensitivities of 1.5 and 1.7 were measured.

The water contents of clayey silt samples ranged from about 16 to 22 per cent with an average water content of 18 per cent. Based on two samples tested, the clayey silt deposits were of low plasticity with average plastic and liquid limits of 17 and 29 per cent, respectively, and an average plasticity index of 12 per cent. The results of the Atterberg Limit testing is shown on Figure A-3 in Appendix A.

The results of two grain size analyses of samples obtained in the clayey silt deposits indicated an average of 36 per cent clay, 43 per cent silt, 18 per cent sand and 3 per cent gravel. The grain size distributions are shown on Figure A-2 in Appendix A.

4.2.6 Silty Clay

Silty clay was encountered in both boreholes advanced for the current investigation. Boreholes from the previous investigation encountered several deposits described as silty clay and clay. For the purposes of this report, soils described as clay in borehole 1 or clay containing 'pieces' of sand or fine gravel in borehole 4 have been interpreted as silty clay.

The clayey silt deposits in boreholes C-1 and C-2 of the current investigation were found to be underlain by silty clay at elevation 168.4 and 171.2 metres, respectively. Based on the inferred stratigraphy, the clayey silt layers encountered in boreholes 1 and 4 of the 1955 investigation were also underlain by silty clay from elevation 168.0 to 171.4 metres. Boreholes C-2, 1 and 4 were terminated in silty clay. Borehole C-1 fully penetrated the silty clay after exploring this stratum for some 26 metres. Shale fragments were noted in the silty clay below elevation 144.0 metres.

The silty clay had standard penetration test N values of 2 to 16 blows per 0.3 metres with an average of 8 blows per 0.3 metres. A single in situ vane test conducted in a softer zone indicated an undrained shear strength of 79 kilopascals (kPa) or a stiff consistency. A sensitivity of 1.5 was measured.

The water contents of the silty clay stratum varied between 20 to 32 per cent with an average water content of 25 per cent. Based on two samples tested, the silty clay deposit is of low to intermediate plasticity with average plastic and liquid limits of 18 and 34 per cent, respectively, and an average plasticity index of 16 per cent. The results of the Atterberg Limit testing is shown on Figure A-3 in Appendix A.

The results of grain size analysis testing of two silty clay samples indicated an average of 42 per cent clay, 45 per cent silt, 12 per cent sand and 1 per cent gravel. The grain size distributions are shown on Figure A-4 in Appendix A.

4.2.7 Bedrock

The bedrock surface was encountered in borehole C-1 of the current investigation at elevation 142.7 metres after penetrating about 40.4 metres of overburden. The shale was explored for 1.4 metres to the termination depth of 41.8 metres with a tricone bit. Fragments of black shale of the Kettle Point formation were retrieved from the rotary drilling.

4.3 Groundwater Conditions

During drilling, groundwater was reported in three of the four boreholes advanced for the previous investigation and both of the boreholes for the current investigation. The groundwater table was not established during drilling at borehole 1. The encountered water levels reported for

the 1955 investigation were 1.8 to 5.8 metres below ground surface, or at about elevations 177 to 181 metres. In the boreholes drilled for the current investigation, groundwater was encountered at about elevations 181 to 182 metres or at depths of 2.0 to 2.4 metres below the ground surface. The encountered water levels and the water level measured in the standpipe in Borehole C-2 are shown on the attached Record of Borehole sheets and are summarized below.

It should be noted that the encountered groundwater levels reported do not indicate the long term stable ground water elevations and that the groundwater levels are subject to seasonal fluctuations. Based on soil colour changes and the encountered and measured groundwater levels, the long term groundwater level has been inferred to be within the fine sand/silty fine sand deposits near elevation 181 metres. The groundwater table may also be influenced by the water level in the adjacent St. Clair River which can vary as much as 1.6 metres. On August 10, 2005, the Canadian Hydrographic Service water level gauge on the St. Clair River at nearby Point Edward, Ontario recorded a high water level of 176.175 metres and a low water level of 175.829 metres as referenced to the International Great Lakes Datum 1985 (IGLD).

BOREHOLE	GROUND SURFACE ELEVATION (m)	WATER LEVEL ELEVATION (m)		DATE
		ENCOUNTERED	MEASURED	
1	183.00	-	-	-
2	182.90	181.11	-	March 14, 1955
3	183.30	180.53	-	March 10, 1955
		177.49	-	March 11, 1955
4	183.20	181.36	-	March 12, 1955
C-1	183.03	181.63	-	July 28, 2005
C-2	182.77	180.79	-	July 22, 2005
		-	180.82	August 3, 2005

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. Michael Arthur and Mr. Lubomir Kosc under the direction of Mr. David J. Mitchell. All routine laboratory testing was conducted at Golder Associates' London laboratory. This laboratory is an accredited participant in the MTO's Soil and Aggregate Proficiency program and is certified for full quality testing of Types C and D Aggregates by the Canadian Council of Independent Laboratories. This report was written by Ms. Dirka U. Prout, P. Eng., a geotechnical engineer under the direction of the Project Manager, Mr. Philip R. Bedell, P. Eng., a Principal with Golder Associates Ltd. 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.
Designated MTO Contact

DUP/PRB/FJH/jk
n:\active\2004\130000\041-130099 urs - gwp 3038 - sarnia\reports\041-130099-7 colborne road\july 28 06 - prelim fdn inv report - colbourne road.doc

PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
PROPOSED COLBORNE ROAD OVERPASS
STRUCTURE WIDENING
HIGHWAY 402, GWP 3038-03-00
AGREEMENT NUMBER 3005-A-000394**

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the draft report provides our recommendations on the foundation aspects for the preliminary design phase of the project. It should be noted that the interpretation of the factual information obtained during the investigation for the existing structures 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 402 will be widened by adding an Exclusive Truck Lane (ETL) in advance of the toll facility for the Blue Water Bridge. The ETL will require the widening and/or replacement of existing Highway 402 bridges in the area. Based on the currently available information, it is understood that the proposed westbound ETL will be accommodated through expansion of the existing Colborne Road Overpass structure to the north. The existing bridge is founded on creosoted timber friction Class B piles which are 9.1 to 12.2 metres long. The piles have a minimum tip diameter of 203 millimetres and a diameter 0.91 metres from the butt which is in the range of 305 to 508 millimetres. The top of the pile caps is approximately at 182.6 metres. The estimated tip elevations vary from 170 to 173 metres.

The approximate elevation of Colborne Road at this location is 183.4 metres. The approximate elevation of the bridge deck is 189.4 metres.

6.2 Bridge Widening Foundations

Based on the results of the current and the 1955 investigations, the subsurface conditions in the area of the widening typically consisted of fill and surficial deposits of topsoil and compact sand and gravel. Substantial fills should be expected at the embankment locations. The coarse granular deposits are underlain by compact silty fine sand, sand and sandy silt from about elevation 181 metres. Beneath the sandy strata, deposits of loose to compact silty sand or very soft to firm clayey silt interlayered with peat and organic silt were encountered. This zone was intercepted near elevation 178 metres near the west abutment and at about elevation 176 metres at the east abutment. The organic deposits were underlain by hard to stiff clayey silt from about elevation 173 metres. Stiff silty clay was found beneath the clayey silt from approximate elevation 171 metres. Black shale bedrock of the Kettle Point formation was encountered at elevation 143 metres in one borehole after penetrating 40 metres of overburden. The groundwater table was encountered near elevation 181 metres near the surface of the fine sands.

On August 3, 2005, the groundwater level was measured at 180.8 metres in the standpipe in borehole C-2.

Various shallow and deep foundation alternatives have been assessed based on the subsurface information noted above and the understanding that the proposed widening will be built with piled foundations and semi-integral abutments. After comparing spread footings, friction piles, and end-bearing piles, it was determined that steel piles driven to practical refusal on bedrock was the preferred founding option for the widening. The risks, consequences, costs and feasibility of the featured options are compared in Table I. The feasibility of the various founding options are discussed below.

6.3 Shallow Foundations

In the vicinity of the proposed widening, organic deposits interlayered with very soft to firm clayey silt are present at 5 to 7 metres below the ground surface. The zone containing the compressible organic deposits is 3 to 5 metres thick and is below the water table. Since removal of this material is not practical due to its depth relative to both the ground surface and groundwater table, spread footings are not considered to be an appropriate founding option for this site. If the widening is constructed using spread footings, extensive preloading and/or use of lightweight fills would be necessary to reduce the post construction settlements. There will also be the potential for differential settlement between the new and existing parts of the structure. Further, the bearing resistance of the native fine sand granular deposits and ease of construction is limited by the relatively shallow water table which during wet seasons of the year would be above the surface of the sand. The estimated cost of shallow footings have been provided in Table I to provide a basis for comparison of deep and shallow foundation alternatives.

6.4 Deep Foundations

The existing structure is founded on treated timber friction piles within the stiff to very stiff clayey silt which have performed adequately for approximately 50 years. However, since semi-integral abutments are proposed, and differential movements between the existing and widened section of the structure are of concern, deep foundations will require a certain degree of flexibility and should provide resistance to settlement. Steel H-piles driven to practical refusal on bedrock have been selected as the preferred foundation alternative. H-piles are recommended because they will readily penetrate the clayey deposits and minimize the amount of disturbance given their shape and small cross-sectional area. They will also have the necessary flexibility required for use with semi-integral abutments and will minimize differential settlement between the existing and widened section.

6.4.1 Geotechnical Axial Resistance – Driven Steel Piles

For preliminary design, the factored axial geotechnical resistance at Ultimate Limit States (ULS) for HP 310 x 110 piles driven to refusal in the shale bedrock at about elevation 143 metres may be taken as 2,000 kilonewtons (kN). This value takes into account the structural capacity of the pile. Vertically driven pile tips should be equipped with Type I driving shoes in accordance with current MTO practice (Standard Ontario Provincial Standard Drawing (OPSD) 3301.00) and battered piles should be equipped with Type II driving shoes to ensure adequate seating of the piles on the bedrock. The surface elevation and quality of the bedrock should be confirmed in the investigation for the final design.

A Serviceability Limit States (SLS) value is not provided because the shale bedrock is considered to be an unyielding material. Under such conditions, SLS values (for 25 millimetres of settlement) do not govern design because the SLS value is much higher than the ULS value.

The pile driving note to be added to the drawings is: “Piles to be driven to bedrock”.

Should the proposed locations of the driven steel piles be in conflict with some existing timber piles, it will be necessary to extract those timber piles.

6.4.2 Downdrag Load (Negative Skin Friction)

Settlement in the granular deposits is expected to occur during construction. Consolidation settlement of the underlying organic and extensive clayey deposits due to the increased loading imposed by the embankments should be anticipated. 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 organic and clayey deposits may take place and settlement of the 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. The abutment piling for the existing structure adjacent to the fills for the new embankments will also be affected by downdrag loads induced by the embankment widening. If the new embankments are constructed well in advance of the piling or lightweight fills are utilized, the downdrag loads on the new piles may be minimized or 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 Canadian Highway Bridge Design Code (CHBDC), and include it as part of the load effects acting on the pile as described in the CHBDC. For preliminary design, the negative skin friction

load on a single end bearing pile may be taken as 200 kN. This value is based on our experience with piles founded in similar soil conditions in the area. The actual negative skin friction will depend on the extent of filling and construction sequencing, both of which are currently unknown. If the embankments are not constructed well in advance of the piling, the downdrag load will have to be reassessed during the detailed design stage by the foundation engineer.

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. Since semi-integral abutments are under consideration, there is a requirement for the piles to move sufficiently to accommodate deflections of the bridge deck.

The abutment piles will be driven through the granular embankment fill and the underlying granular and cohesive soils. The resistance to lateral loading may be based on the following assessed values:

SOIL TYPE	HORIZONTAL RESISTANCE VALUES (kN) PER PILE	
	Factored ULS	SLS
Embankment fill (granular)	120	35
Granular deposits	150	50
Organic deposits	30	10
Clayey silt and silty clay deposits	160	65

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, $d = \text{Pile Diameter}$</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 abutment additions 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 Ontario Provincial Standard Specifications (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 Ontario Provincial Standard Drawing (OPSD) 3501.00 and 3504.00.
- A compaction surcharge equal to 12 kPa 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:

	<u>GRANULAR A</u>	<u>GRANULAR B</u>
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 and OPSD 3504.00.

6.6 Embankments

The new embankments for the proposed overpass widening are expected to be up to 6 metres in height. The actual extent of the additional fills required is currently unknown. The widened embankments should be benched into the existing embankments in accordance with OPSD 202.010. 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 embankments constructed with suitable native or borrow materials.

The topsoil and any surficial organic materials should be removed from within the area of the embankment and the exposed subgrade soils should be proofrolled prior to fill placement and the new fills benched into the existing abutment in accordance with OPSD 202.010.

Construction of the embankment widening above the prepared subgrade may be carried out using clean earth fill (in accordance with OPSS 212) or select subgrade material (in accordance with OPSS 1010) depending on material availability. All embankment fill should be placed in regular lifts and compacted.

Embankment settlements will be dependent on the extent of the additional filling required. Settlements could be reduced by preloading and/or using lightweight fill. Preliminary estimates of total embankment settlements primarily within the organic and cohesive deposits are about 390 millimetres in the centre of the embankment widening and 160 millimetres at the toe. Additional settlement of the granular deposits and existing granular fill is expected to occur during construction and is estimated to be in the order of 30 to 40 millimetres total.

Primary settlements prior to paving further can be reduced by preloading if time is available. Settlements could be reduced by using lightweight fill. A more detailed settlement analysis should be conducted once the construction sequencing is known and the design has been finalized.

6.7 Excavations and Temporary Cut Slopes

Excavations for pile cap construction will likely encounter the surficial fill and granular deposits above the groundwater table. Temporary open cut slopes should be maintained no steeper than 1 horizontal to 1 vertical.

Surficial water seepage into the excavations should be expected, and will be heavier during periods of sustained precipitation. Surface water runoff should be directed away from the excavations at all times. Pumping from well 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. Significant groundwater seepage may occur if the groundwater table is high during construction or excavations extend below elevation 180.8 metres. Dewatering will be necessary if the groundwater table is encountered during excavation of the granular deposits. The appropriate Non Standard Special Provisions (NSSP) should be included in the contract documents.

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 granular materials beneath the groundwater table and any fills encountered at this site would be classified as Type 3 soils. The compact granular soils above the groundwater table and the underlying cohesive deposits would be classified as Type 2 soils.

Roadway protection should conform to Performance Level 2, SP No. 539S01.

7.0 MISCELLANEOUS

This report was written by Ms. Dirka U. Prout, P. Eng., a geotechnical engineer under the direction of the Project Manager, Mr. Philip R. Bedell, P. Eng., a Principal with Golder Associates Ltd. 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.
Designated MTO Contact

DUP/PRB/FJH/jk
n:\active\2004\130000\041-130099 urs - gwp 3038 - samia\reports\041-130099-7 colborne road\july 28 06 - prelim fdn inv report - colbourne road.doc

TABLE I
COMPARISON OF FOUNDATION ALTERNATIVES
COLBORNE ROAD OVERPASS STRUCTURE WIDENING
G.W.P. 3038-03-00

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risks/Consequences
Spread footings supported on native sand and sand and gravel	<ul style="list-style-type: none"> • Not considered feasible due to low allowable bearing value and presence of peat at depth • Existing structure founded on piles 	<ul style="list-style-type: none"> • Least Expensive option 	<ul style="list-style-type: none"> • Time and cost of settlement mitigation measures • Even if mitigation adopted, settlement of shallow foundations could still take place 	<ul style="list-style-type: none"> • Estimated cost \$30, 000 assuming two 5 m wide strip footings (if dewatering not required) • Expected to be less expensive than deep foundation options • Cost of settlement mitigation measures not included 	<ul style="list-style-type: none"> • Even if mitigation in place, shallow foundations may still be affected by settlement of underlying peat and clayey deposits • Loosening of subsoils if groundwater level is high at the time of footing construction • Potential for differential settlement between widening and existing structure which is founded on piles
End bearing steel H-pile foundations driven to shale bedrock	<ul style="list-style-type: none"> • Feasible for support of all foundation elements 	<ul style="list-style-type: none"> • High bearing resistance • Negligible settlement 	<ul style="list-style-type: none"> • Possibility of pile tip damage during driving in rock • Care must be taken with driving of battered piles to ensure that they do not deflect along the bedrock surface 	<ul style="list-style-type: none"> • Estimated cost \$130, 000 (pile installation only) • More expensive than shallow foundations but preferred technical solution 	<ul style="list-style-type: none"> • Possible pile tip damage if piles are not adequately protected while driving in bedrock

NOTES:

- 1) Costs are very preliminary estimates and are intended to provide a comparison between alternatives rather than actual construction costs.
- 2) Table to be read in conjunction with accompany report.

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), 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.) split spoon sampler for a distance of 300 mm (12 in.)

Consistency

	<u>kPa</u>	<u>psf</u>
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

(b) Cohesive Soils

Dynamic Cone Penetration Resistance; N_d :

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 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. General

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity index $= (w_l - w_p)$
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
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

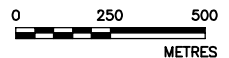
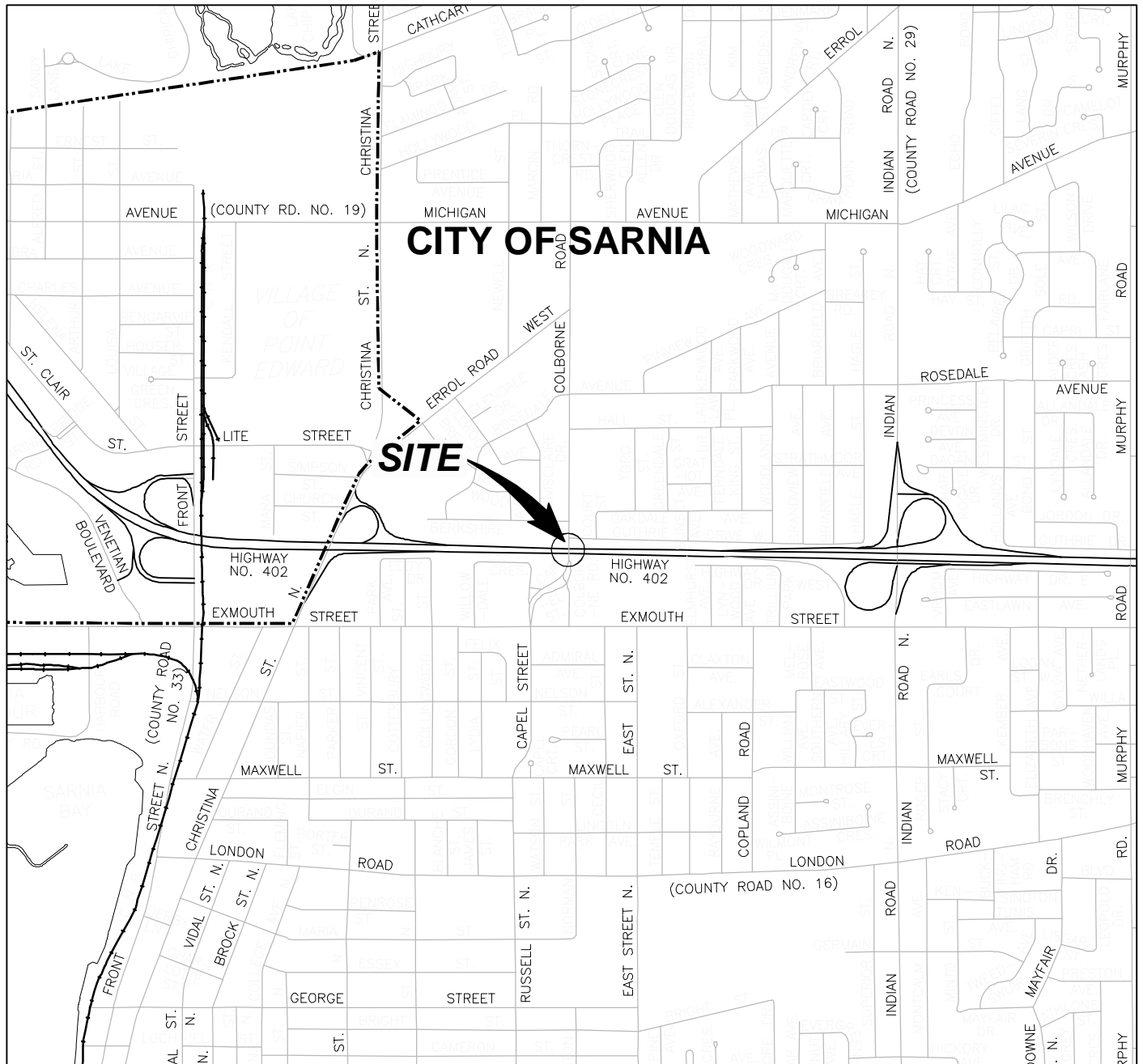
(c) Consolidation (one-dimensional)


C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

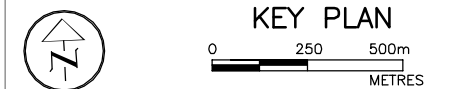
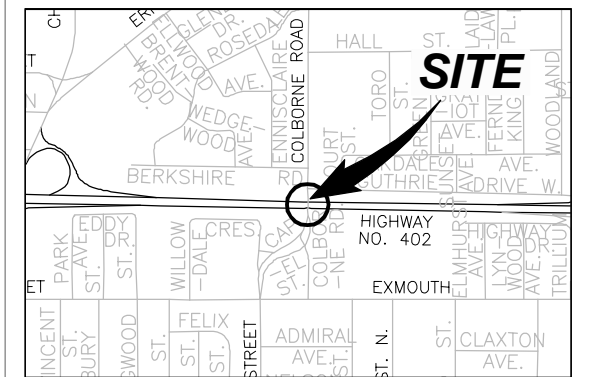
(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity







- Notes:**
- 1 $\tau = c' + \sigma' \tan \phi'$
 - 2 shear strength = (compressive strength)/2
 - * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)



PROJECT				COLBORNE ROAD OVERPASS WIDENING WP No. 3038-03-00 HWY. 402			
TITLE				SITE LOCATION PLAN			
 Golder Associates LONDON, ONTARIO		PROJECT No.		FILE No.		SCALE	
		041-130099-7		041130099-7F001		AS SHOWN	
CADD		DCH	Aug 29/05	REV.		0	
CHECK						FIGURE 1	



LEGEND

-  Borehole (Current Investigation)
 Borehole (Previous Investigation)
 Seal
 Piezometer
 N Blows/0.3m (Std. Pen. Test, 475 j/blow)
 DRY Borehole dry during drilling
 WL in piezometer
 WL upon completion of drilling

No.	ELEVATION (metres)	CO-ORDINATES	
		NORTH	EAST
OTHERS			
1	183.00	4 760 883.6	313 862.1
2	182.90	4 760 882.9	313 883.4
3	183.30	4 760 847.1	313 860.8
4	183.20	4 760 846.4	313 882.1
GOLDER			
C-1	183.03	4 760 839.1	313 858.0
C-2	182.77	4 760 896.0	313 885.2

NOTES

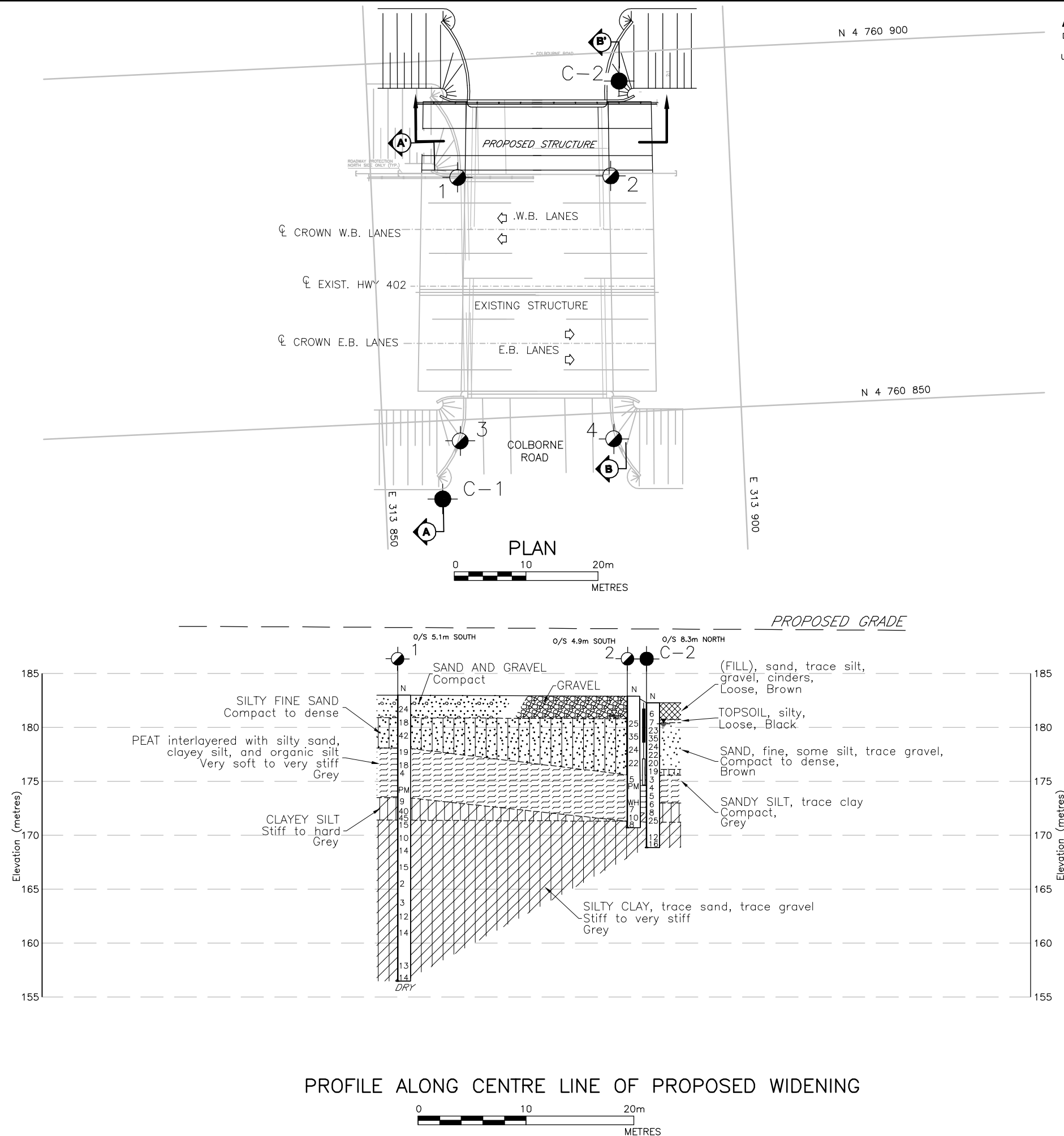
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

This drawing is for subsurface information only. The proposed structure details are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

REFERENCE

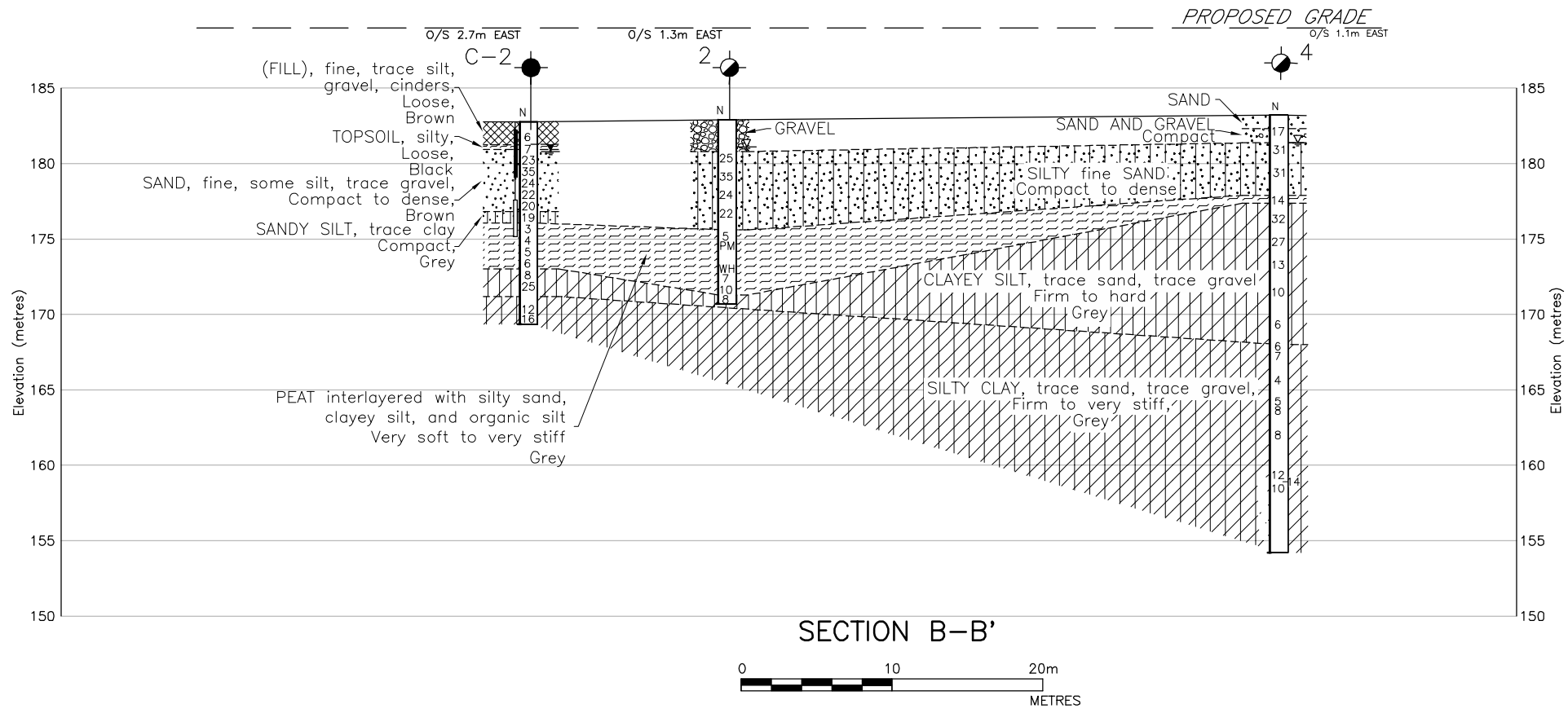
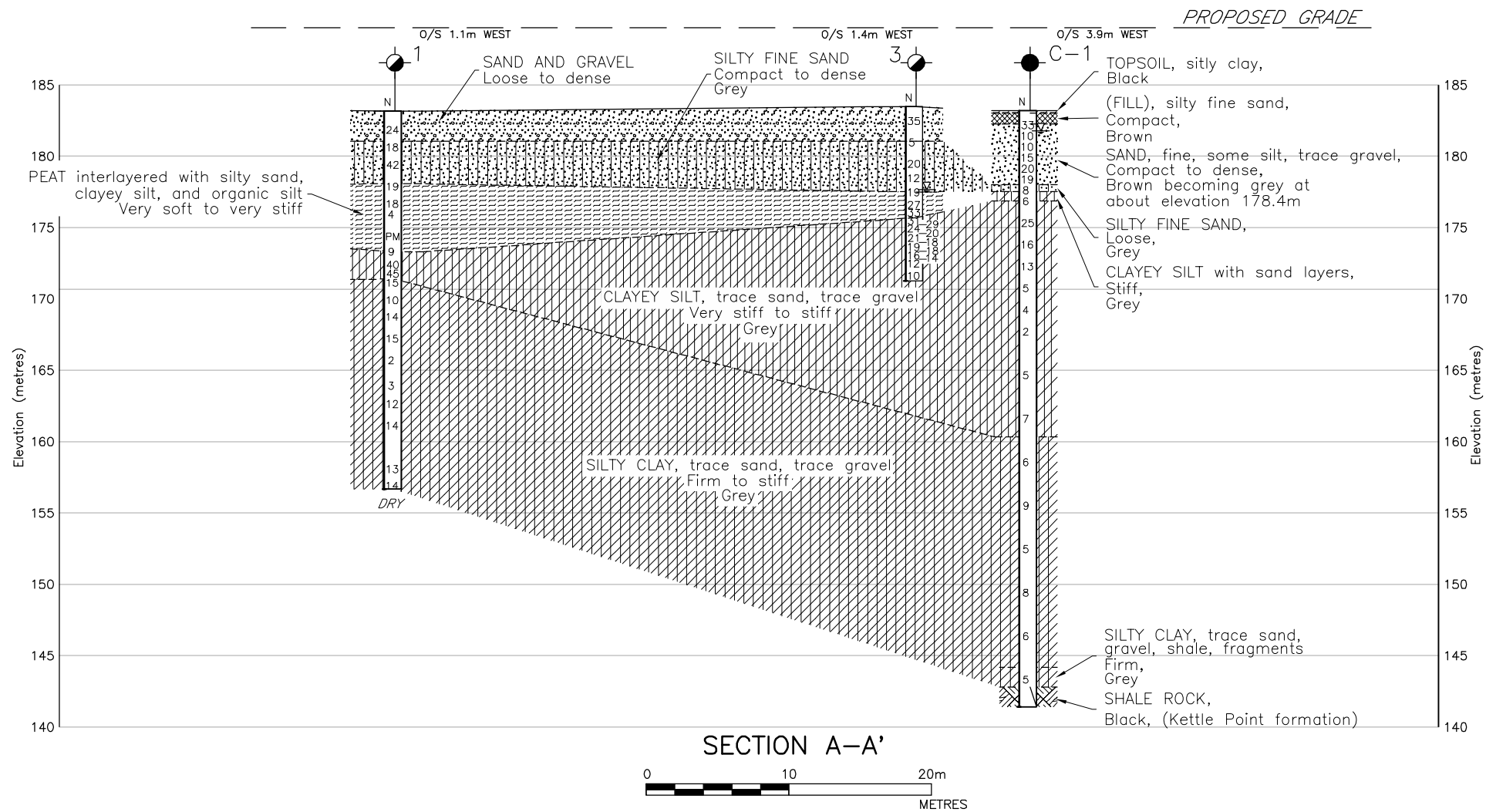
- 1) REFERENCE : URS
ENTITLED: GENERAL ARRANGEMENT COLBORNE ROAD OVERPASS
SITE No. : 14-38
DATED: JUNE 2005
- 2) BOREHOLE STRATIGRAPHY AS NOTED ON RECORDS OF BOREHOLES
FOR DEPARTMENT OF HIGHWAYS ONTARIO REPORT No. 55-F-212C

NO.	DATE	BY	REVISION	
Geocres No. 40J16-68				
HWY. No. 40Z		PROJECT NO.: 041-130099-7		
SUBM'D.	-	CHKD:	DATE: July 20/06	
DRAWN: DCH		CHKD.	APPD.	DWG. 1



1 = 1 metric
D size dwg 22" x 32" 11" x 17" plot half scale

041130099-7D002.dwg



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

DIST 1 HWY. 402
CONT. No.
WP No. 3038-03-00

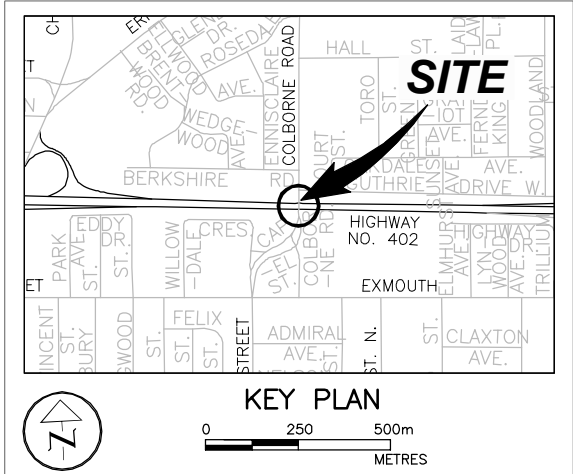


COLBORNE ROAD
OVERPASS WIDENING
SOIL STRATA

SHEET



Golder Associates Ltd.
LONDON, ONTARIO, CANADA



LEGEND

- Borehole (Current Investigation)
- Borehole (Previous Investigation)
- Seal
- Piezometer
- N Blows/0.3m (Std. Pen. Test, 475 j/blow)
- DRY Borehole dry during drilling
- WL in piezometer
- WL upon completion of drilling

No.	ELEVATION (metres)	CO-ORDINATES	
		NORTH	EAST
OTHERS			
1	183.00	4 760 883.6	313 862.1
2	182.90	4 760 882.9	313 883.4
3	183.30	4 760 847.1	313 860.8
4	183.20	4 760 846.4	313 882.1
GOLDER			
C-1	183.03	4 760 839.1	313 858.0
C-2	182.77	4 760 896.0	313 885.2

NOTES

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

This drawing is for subsurface information only. The proposed structure details shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents

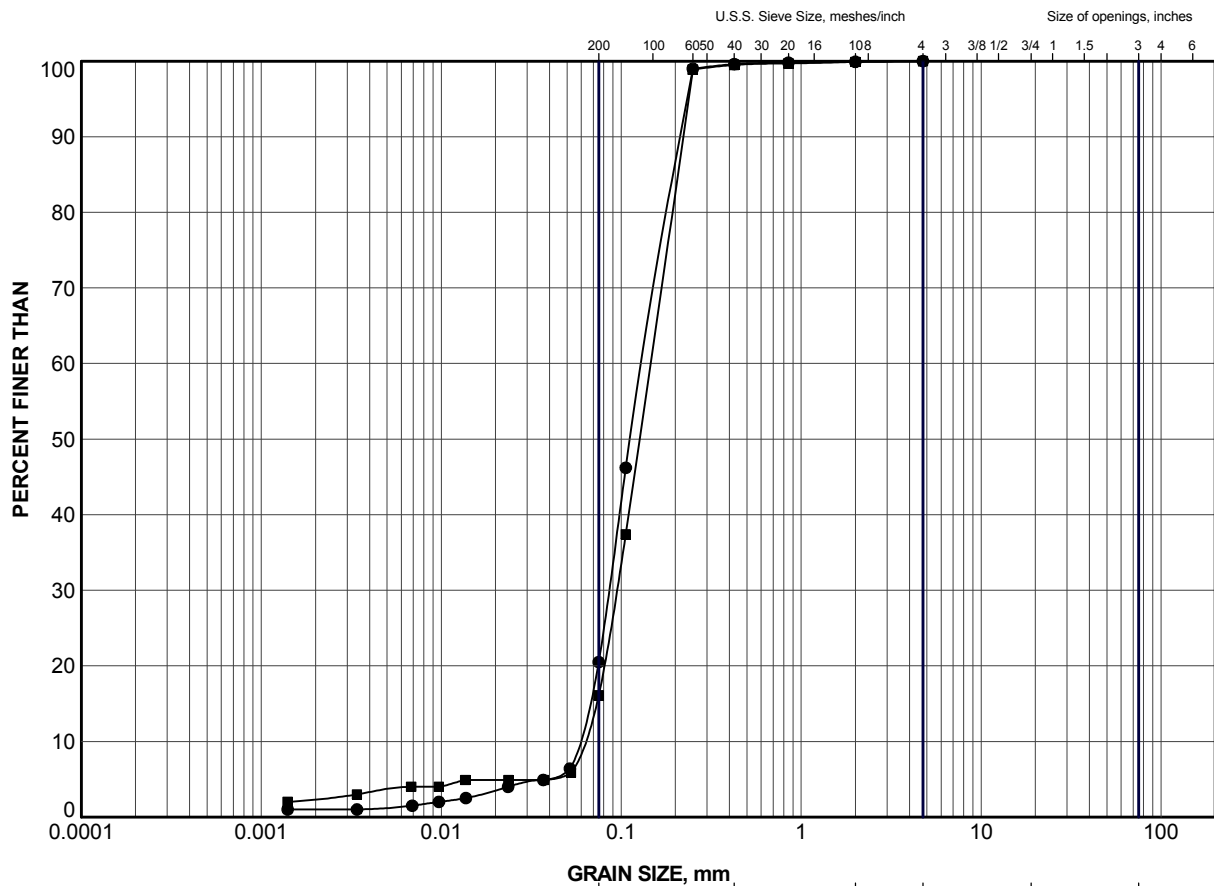
REFERENCE

- REFERENCE : URS
ENTITLED: GENERAL ARRANGEMENT COLBORNE ROAD OVERPASS
SITE No. : 14-38
DATED: JUNE 2005
- BOREHOLE STRATIGRAPHY AS NOTED ON RECORDS OF BOREHOLES FOR DEPARTMENT OF HIGHWAYS ONTARIO REPORT No. 55-F-212C

NO.	DATE	BY	REVISION
Geocres No. 40J16-68			
HWY. No. 402		PROJECT NO.: 041-130099-7	
SUBM'D.	-	CHKD:	DATE: July 20/06
DRAWN: DCH		CHKD.	APPD. DWG. 2

APPENDIX A


LABORATORY TEST RESULTS

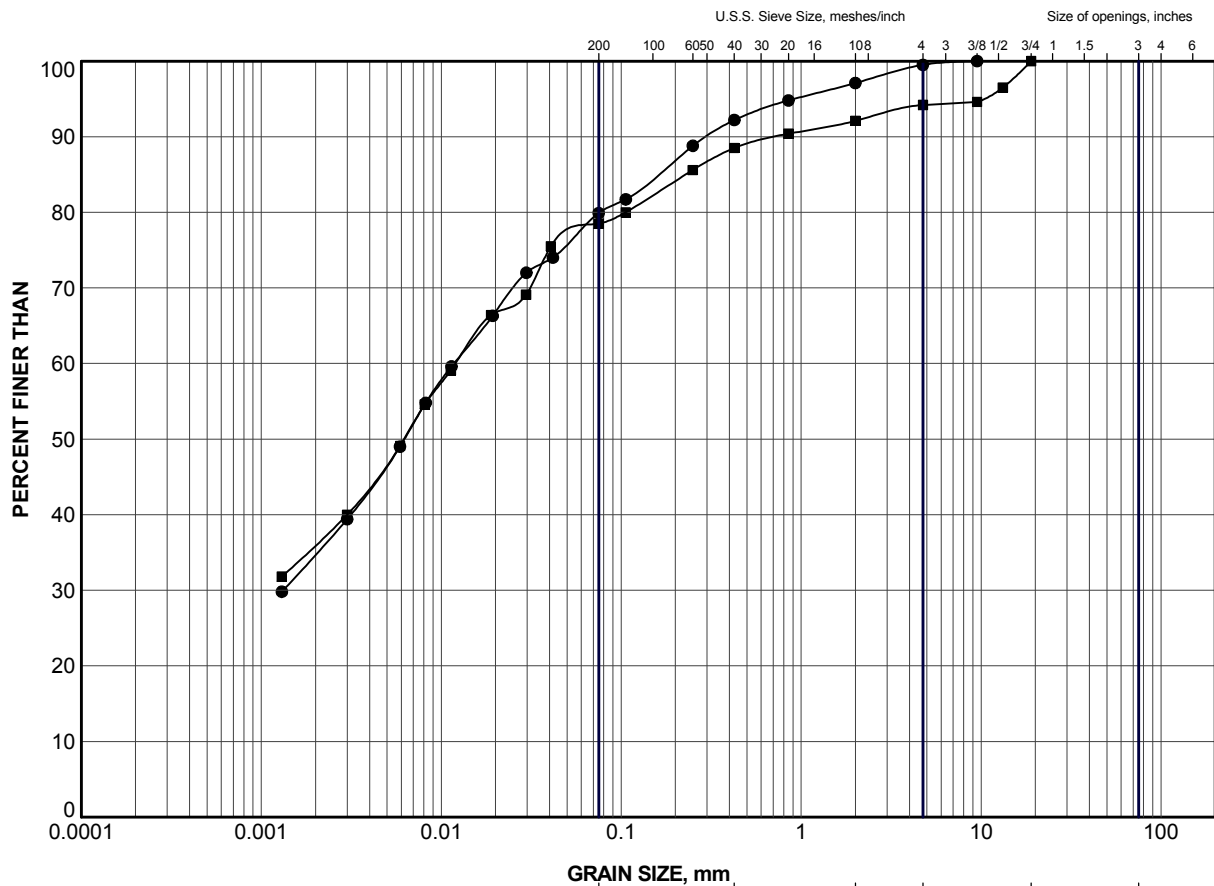


CLAY AND SILT		SAND SIZE, mm					Cobble Size
		fine	medium	coarse	fine	coarse	
		SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C-1	5	179.0
■	C-2	6	178.0


PROJECT					COLBORNE ROAD OVERPASS WIDENING WP No. 3038-03-00 HWY. 402				
TITLE					GRAIN SIZE DISTRIBUTION SAND (fine)				
PROJECT No.		041-130099-7		FILE No.		041-130099-7.GPJ			
DRAWN		DCH		Aug 31/05		SCALE		N/A	
CHECK						REV.			
 Golder Associates LONDON, ONTARIO					FIGURE A-1				

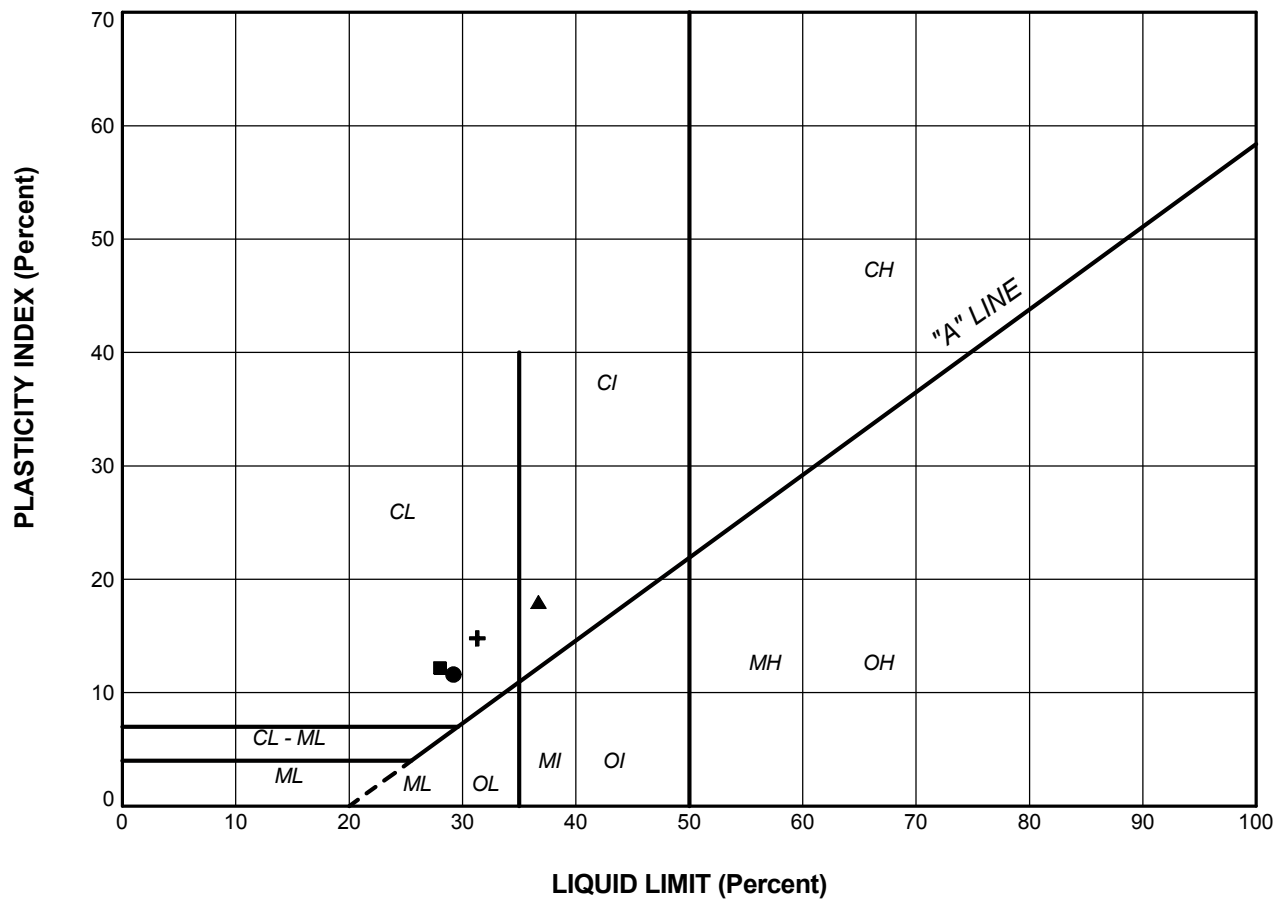


CLAY AND SILT	SAND SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

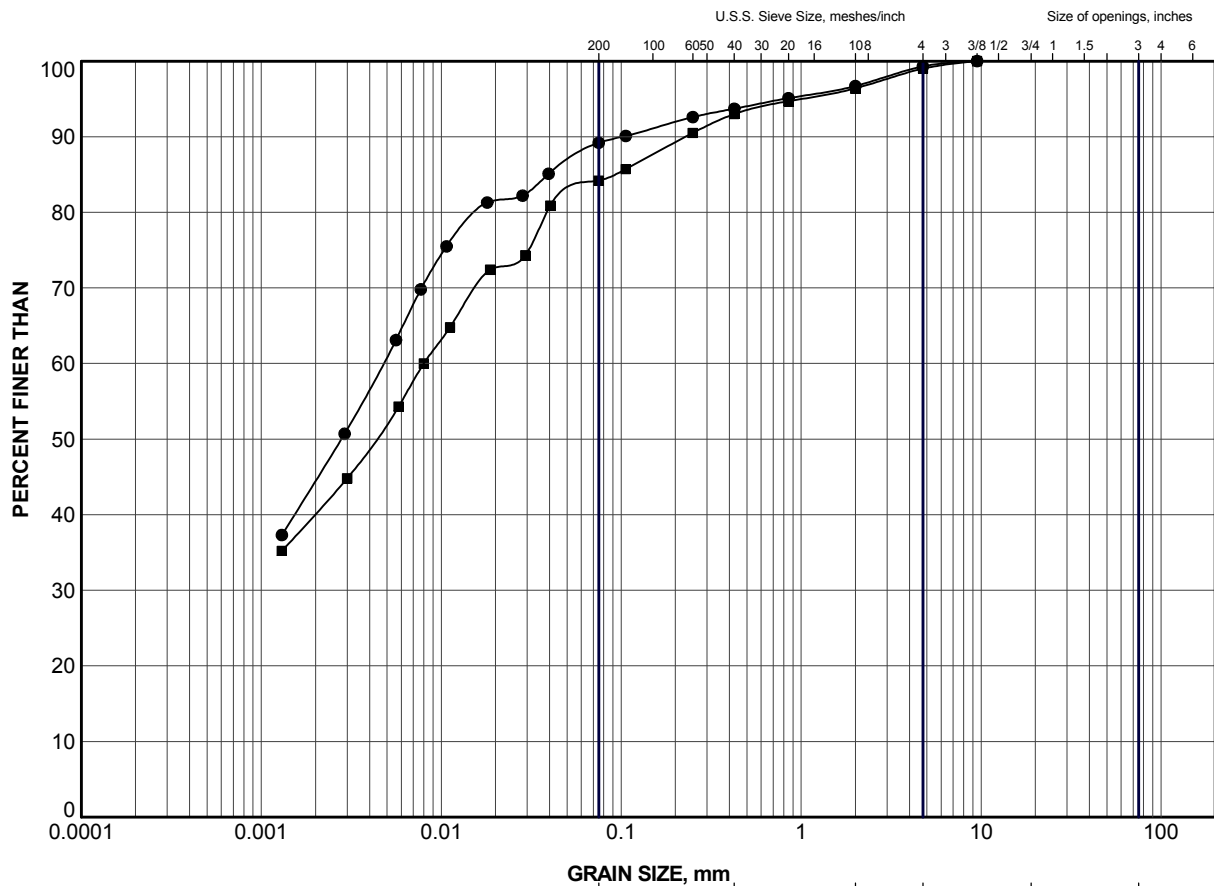
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C-1	9	175.2
■	C-1	12	170.6

PROJECT				COLBORNE ROAD OVERPASS WIDENING WP No. 3038-03-00 HWY. 402			
TITLE				GRAIN SIZE DISTRIBUTION CLAYEY SILT			
PROJECT No.		041-130099-7		FILE No.		041-130099-7.GPJ	
DRAWN		DCH		SCALE		N/A	
CHECK				REV.			
		Aug 31/05					
 Golder Associates LONDON, ONTARIO				FIGURE A-2			



PROJECT				COLBORNE ROAD OVERPASS WIDENING WP No. 3038-03-00 HWY. 402			
TITLE							
PLASTICITY CHART							
PROJECT No.		041-130099-7		FILE No.		041-130099-7.GPJ	
DRAWN	DCH	Aug 31/05		SCALE	N/A	REV.	
CHECK				FIGURE A-3			





APPENDIX B

RECORDS OF PREVIOUS BOREHOLES

SOIL INVESTIGATIONS AND TESTING LTD.

5 YONGE STREET SOUTH
BOX 747
RICHMOND HILL, ONTARIO

Job Name COLBORNE ROAD OVERPASS CAH # 402, SARNIA, ONTARIO.

Order No. SI-55/9

Client PROCTOR, REDFERN & LAUGHLIN, (PROJECT - NO. E.O. 556)
DEPARTMENT OF HIGHWAYS, (WORK ORDER NO. 4-1081)

Borehole No. 1 Diameter 2½" Date MARCH 15th - 17th/55

Borehole Location Refer to plot plan Elevation 600.3'

Description	Elevation	Legend	SAMPLE	Depth	Thickness	Blows Split- tube	Depth To Water Below Ground Level
				0'-0"			140 lb. hammer 30" drop
Fine to coarse sand & fine to coarse gravel (moist)				3'-0"			
Medium dense grey fine to coarse sand & fine to coarse gravel (wet)			1	4'-0"	1'-0"	24	
As sample #1							
Medium grey silty sand (wet)			2	7'-0"	1'-0"	18	
As sample #2				8'-0"			
Dense grey fine silty sand (wet)			3	11'-0"	1'-0"	42	
As sample #3				12'-0"			
Medium dense grey fine silty sand containing decayed vegetation (wet)			4	16'-0"	1'-0"	19	
As sample #4				17'-0"			
Medium grey fine silty sand (wet)			5	20'-0"	1'-0"	18	
Grey clayey silt (moist)				21'-0"			
Very soft grey clayey silt and decayed vegetation (moist)			6	22'-0"	1'-0"	4	
As sample #6				24'-0"			
As sample #6				25'-0"			
				28'-0"	3'-0"	PUSHED SAMPLER	
Medium stiff grey peat with slight clay content & decayed vegetation (moist)			7	31'-0"	1'-0"	9	
Soft grey silty clay (moist)				32'-0"			
Hard greyish brown silty clay containing fine to medium gravel (slightly moist)			8	34'-0"	1'-0"	40	
As sample #8				35'-0"	1'-0"	45	
As sample #8				36'-0"			
Stiff grey clay containing coarse sand (moist)			9	38'-0"	1'-0"	15	
As sample #9				39'-0"			
Medium stiff grey clay containing coarse sand (moist)				42'-0"	1'-0"	10	
No change				43'-0"			
Stiff grey plastic clay containing fine gravel (moist)			10	46'-0"	1'-0"	14	
				47'-0"			

CONTINUED

SOIL INVESTIGATIONS ~~XXXXXXXXXXXX~~ LTD.





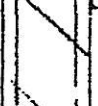





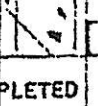

5 YONGE STREET SOUTH
~~XXXXXXXXXXXXXXXXXXXXXXXXXXXX~~
 BOX 717
 RICHMOND HILL, ONTARIO

Job Name COLBORNE ROAD OVERPASS CAH #402, SARNIA, ONTARIO.
 PROCTOR, REDFERN & LAUGHLIN, (PROJECT - NO. E.O. 556)
 Client DEPARTMENT OF HIGHWAYS, (WORK ORDER NO. 4-1081)

Order No. 51-55/9

Borehole No. 1 Diameter 2 1/2" Date MARCH 17th-18th/55

Borehole Location Refer to plot plan. Elevation 600.3'

Description	Elevation	Legend	SAMPLE	Depth	Thickness	Blows Split tube	Depth To Water Below Ground Level
		CONTINUED					140 lb. hammer 30" drop
Stiff gray plastic clay containing fine gravel (moist) No change				47'-0"			
Stiff gray plastic clay containing 2" seam of gray fine silty sand (moist)			11	51'-0" 52'-0"	1'-0"	15	
Gray plastic clay (moist)							
Very soft gray clay with slight silt content (moist)			12	56'-0" 57'-0"	1'-0"	2	
As sample #12							
As sample #12				61'-0" 62'-0"	1'-0"	3	
As sample #12							
Medium stiff gray clay with a slight silt content (moist)			13	66'-0" 67'-0"	1'-0"	12	
As sample #13							
Stiff grayish blue clay (moist)			14	71'-0" 72'-0"	1'-0"	14	
As sample #14							
Stiff grayish blue clay with slight silt content containing fine gravel (moist)			15	81'-0" 82'-0"	1'-0"	13	
As sample #15							
As sample #15				86'-0" 87'-0"	1'-0"	14	
		HOLE COMPLETED					

5 YONGE STREET SOUTH
RICHMOND HILL, ONTARIO

Order No. 51-55/9

Borehole No. 2 Diameter $2\frac{1}{2}"$ Date MARCH 12th/55

Elevation 600.2'

Description	Elevation	Legend	SAMPLE	Depth	Thickness	Blows Split- tube	Depth To Water Below Ground Level
				0'-0"			140 lb. hammer 30" drop
Fine to coarse gravel (moist)					7'-0"		
Medium dense grey fine silty sand (highly saturated) As sample #1			1	7'-0" 8'-0"	1'-0"	25	
Dense grey fine silty sand (highly saturated) As sample #2			2	11'-0" 12'-0"	1'-0"	35	
Medium dense grey fine silty sand (wet) As sample #3 As sample #3			3	15'-0" 16'-0"	1'-0"	24	
				19'-0" 20'-0"	1'-0"	22	
Soft grey clayey silt (moist)			4	24'-0" 25'-0"	1'-0"	5	
Very soft grey silty clay (moist)					5'-0"	PUSHED	SAMPLER
Very soft grey silty clay (moist)				30'-0"			
					3'-0"	PUSHED	SAMPLER
Soft brown peat containing thin seams of gray fine silty sand (moist) As sample #5			5	33'-0" 34'-0"	1'-0"	7	
Medium stiff grey silty clay containing seams of coarse grey sand (moist)				38'-0" 39'-0" 40'-0"	1'-0" 1'-0"	10 8	
HOLE COMPLETED							

6'

↑ WATER RISE 8 AM MARCH 14th

↓ 12'

5 YONGE STREET SOUTH
BOX 747
RICHMOND HILL, ONTARIO

Order No. SI-55/9

Borehole Location Refer to plot plan.

Elevation 601.3'

Description	Elevation	Legend	DEPTH	Thickness	Blows SPIT- TUBE	Depth To Water Below Ground Level
						140 lb. Hammer 30" drop
Gray fine to coarse sand & coarse gravel (moist)		4	0'-0"	3'-0"		
Dense gray fine to medium sand & coarse gravel (moist)		4	3'-0" 4'-0"	1'-0"	35	
As sample #1						
Loose gray fine sand containing fine to coarse gravel (wet)			7'-0" 8'-0"	1'-0"	5	
Gray fine silty sand (highly saturated)						9'-0" 12:30 PM 10' BAILED DRY 12 NOON
Medium dense gray fine silty sand (highly saturated)			11'-0" 12'-0"	1'-0"	20	
Gray fine silty sand (wet)						
Medium gray fine silty sand (wet)			15'-0" 16'-0"	1'-0"	12	
As sample #4						
Very stiff gray silty clay containing decayed vegetation (moist)			19'-0" 20'-0"	1'-0"	19	19' WATER RISE 8 AM MARCH 11th
Gray silty clay (moist)						
Gray silty clay containing decayed vegetation (jar sample)			22'-6" 23'-6" 24'-4" 25'-0"	1'-0" 0'-10"	27 33	2" SHELBY-TUBE
Hard grayish brown silty clay containing fine gravel (slightly moist)			26'-0" 27'-0" 28'-0" 29'-0" 30'-0" 31'-0"	1'-0" 1'-0" 1'-0" 1'-0" 1'-0" 1'-0"	31 24 24 20 21 18	26'
Very stiff grayish brown clay containing coarse sand and fine gravel (moist)			32'-0" 33'-0" 34'-0" 35'-0" 36'-0"	1'-0" 1'-0" 1'-0" 1'-0" 1'-0"	19 18 16 14 12	
Stiff gray silty clay with coarse sand content (moist)						
As sample #9						
As sample #9						
Medium stiff gray silty clay (moist)			39'-0" 40'-0"	1'-0"	10	

HOLE COMPLETED

SOIL INVESTIGATIONS ~~XXXXXXXXXXXX~~ LTD.

5 YONGE STREET SOUTH
~~XXXXXXXXXXXXXXXXXXXX~~
 BOX 747
 RICHMOND HILL, ONTARIO

Job Name COLBORNE ROAD OVERPASS CAH #402, SARNIA, ONTARIO.
 Client PROCTOR, REDFERN & LAUGHLIN, (PROJECT - NO. E.O. 556)
 DEPARTMENT OF HIGHWAYS, (WORK ORDER NO. 4-1081)

Order No. SI-55/9

Borehole No. 4 Diameter 2½" Date MARCH 11th/55

Borehole Location Refer to plot plan Elevation 601.0'

Description	Elevation	Legend	SAMPLE	Depth	Thickness	Blows Split- tube	Depth To Water Below Ground Level
				0'-0"	3'-0"		140 lb. hammer 30" drop
Grey fine sand (moist)							
Medium grey fine to coarse sand & fine to medium gravel (wet)			1	3'-0" 4'-0"	1'-0"	17	
As sample #1							
Grey fine silty sand (wet)				6'-0"			6'-0"
Dense grey fine silty sand (wet)			2	7'-0" 8'-0"	1'-0"	31	
As sample #2							
Dense grey fine silty sand (highly saturated)			3	12'-0" 13'-0"	1'-0"	31	
As sample #3							
Lost sample				16'-0" 17'-0"	1'-0"	28	
As sample #3				17'-6" 18'-0"			
Stiff grey silty clay containing decayed vegetation (moist)			4	18'-0" 19'-0"	1'-0"	14	
Greyish brown silty clay containing coarse sand & fine gravel (slightly moist)							
Hard greyish brown clay containing coarse sand & fine gravel (slightly moist)			5	22'-0" 23'-0" 24'-0" 25'-0" 26'-0"	1'-0" 1'-0" 1'-0" 1'-0" 1'-0"	32 33 31 30	
As sample #5				27'-0"	1'-0"	27	
Very stiff greyish brown clay con- taining silt content, coarse sand & fine gravel (moist)			6	27'-0" 28'-0" 29'-0"	1'-0" 1'-0" 1'-0"	27 27 24	
As sample #6 becoming softer with depth							
Stiff greyish brown clay contain- ing fine to medium gravel (moist)			7	32'-0" 33'-0"	1'-0"	13	
As sample #7							
Stiff grey clay with slight sand content (moist)			8	36'-0" 37'-0" 38'-0" 39'-0" 40'-0"	1'-0" 1'-0" 1'-0" 1'-0" 1'-0"	11 12 10 11	
HOLE COMPLETED							

WATER
RISE
8 AM
MARCH 12th

SOIL INVESTIGATIONS AND TESTING LTD.

5 YONGE STREET SOUTH
XXXXXXXXXXXXXXXXXXXX
BOX 747
RICHMOND HILL, ONTARIO

Job Name COLBORNE ROAD OVERPASS CAH #1102, SARNIA, ONTARIO.

Order No. 51-55/9

PROCTOR, REDFERN & LAUGHLIN, (PROJECT - NO. E.O. 556)
Client DEPARTMENT OF HIGHWAYS, (WORK ORDER NO. 4-1081)

Drillhole No. 4 Diameter 2 1/2" CONTINUED

Date MARCH 28th - 30th/55

Drillhole Location Refer to plot plan.

Elevation 601.0'

Description	Elevation	Legend	Depth	Thi	Blows split-	Depth To Water Below Ground Level
No sampling or soil classifications required			40'-0"	5'-0"	tube	140 lb. hammer 30" drop
Soft grey clay containing coarse sand & fine gravel (moist)			45'-0" 46'-0"	1'-0"	6	NO FREE WATER @ 46'
As sample #1						
Soft grey clay containing pieces of coarse sand & fine gravel (moist)			50'-0" 51'-0" 52'-0"	1'-0" 1'-0"	6 7	2" SHELBY TUBE
As sample #2 with pieces of medium gravel						
Soft grey clay containing pieces of coarse sand (moist)			57'-0" 58'-0"	1'-0"	4	
As sample #3						
Soft grey clay (moist)			62'-0" 63'-0" 64'-0"	1'-0" 1'-0"	5 8	2" SHELBY TUBE
Soft grey clay containing pieces of coarse sand & fine to medium gravel (moist)						
No change			69'-0" 70'-0"	1'-0"	8	
No change						
Medium stiff grey clay containing pieces of coarse sand & fine gravel (moist)			75'-0" 76'-0"	1'-0"	12	HOLE DRY @ 76'
Medium stiff grey clay containing pieces of fine to medium & coarse gravel (moist)			79'-0" 80'-0" 81'-0"	1'-0" 1'-0"	12 14	HOLE DRY @ 81'
Medium stiff grey clay containing pieces of fine to medium gravel (moist)			82'-0" 83'-0" 84'-0"	1'-0" 1'-0"	10 15 20	
Drove 2" cross chopping bit			85'-0" 86'-0" 87'-0"	1'-0" 1'-0"	36 48 48	
" " " " " "			88'-0" 89'-0" 90'-0"	1'-0" 1'-0"	57 59 63	
" " " " " "			91'-0" 92'-0" 93'-0"	1'-0" 1'-0"	61 77 72	
" " " " " "			94'-0" 95'-0"	1'-0" 1'-0"	92 93	

- Same as sample #7

HOLE COMPLETED

made by driving
dynamic cone

APPENDIX C
SITE PHOTOGRAPHS

Colborne Road Overpass



Photo 1 : Undated photograph¹ shows pile installation at an Ontario Department of Highways overpass in Sarnia. The site is believed to be Colborne Road. Vertical and battered jack pine piles were driven to found the abutments.

1) Pressure Treated Timber Piles: A Manual for Architects and Engineers (1st ed.). (1962). Photograph (pp.43).
Ottawa, ON: Canadian Institute of Timber Construction

Colborne Road Overpass



Photo 2 : Colborne Road Overpass looking north along centre line of southbound lane of Capel Street.



Photo 3 : Colborne Road Overpass looking south along Colborne Road.