Golder Associates Ltd.

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

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT ESSEX COUNTY ROAD 19 AND CPR, ESSEX COUNTY ROAD 19 AND ESSEX COUNTY ROAD 22 AND ESSEX COUNTY ROAD 22 AND LESPERANCE ROAD GRADE SEPARATIONS ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING GWP 3031-06-00, AGREEMENT NO. 3005-E-0028 MINISTRY OF TRANSPORTATION - SOUTHWESTERN REGION

Submitted to:

Dillon Consulting Limited
130 Dufferin Avenue, Suite 1400
London, Ontario
N6A 5R2

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TABLE OF CONTENTS

SECTION PAGE

PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

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ESSEX COUNTY ROAD 19 AND CPR, ESSEX COUNTY ROAD 19 AND ESSEX COUNTY ROAD 22 AND ESSEX COUNTY ROAD 22 AND LESPERANCE ROAD GRADE SEPARATIONS
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
GWP 3031-06-00, AGREEMENT NO. 3005-E-0028
MINISTRY OF TRANSPORTATION - SOUTHWESTERN REGION

1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the preliminary design work for GWP 3031-06-00. The project involves the preliminary design for widening of a 13 kilometre section of Essex County Road 19 (Manning Road) from 2 to 4 lanes from Highway 3 northerly to the Canadian National Railway (CNR). The project also includes the widening of a 3 kilometre section of Essex County Road 22 from 4 to 6 lanes from the City of Windsor limit (east of Banwell Road) to Lakeshore Boulevard. Improvements to be implemented in conjunction with this project consist of:

- Highway 401 interchange improvements;
- Intersection improvements at all intersections; and,
- Analysis of grade separation/interchange options at the intersections of County Road 19 with the CPR tracks as well as with County Road 22 and the County Road 22/Lesperance Road intersection.

This report addresses the preliminary foundation design for the grade separations at:

Site A - Canadian Pacific Railway Tracks & County Road 19 (Manning Road) - MTO Site B-19-04
Site B - County Road 19 (Manning Road) and County Road 22 (Highway 2) - MTO Site B-22-03
Site C - County Road 22 (Highway 2) & Lesperance Road - MTO Site 22-02

The purpose of the preliminary foundation investigation is to determine the subsurface conditions at the locations of the proposed works by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal and in Golder Associates' proposal P61-3048 dated June 30, 2006. The work was carried out in accordance with our revised Quality Control Plan for Foundations Engineering dated October 27, 2006 and our letter regarding preliminary engineering services for this project dated August 22, 2007 (revised September 7, 2007).

Dillon provided preliminary general arrangement and construction staging drawing in digital format.

2.0 SITE DESCRIPTIONS

2.1.1 Project Area

GWP 3031-06-00 extends along Essex County Road 19 (Manning Road) from Highway 3 to the Canadian National Railway/VIA track 13 kilometres to the north and consists of widening Manning Road from 2 lanes to 4 lanes. The project also includes the widening of County Road 22 from 4 to 6 lanes from the east limit of the City of Windsor (east of Banwell Road) for three kilometres to the east, Highway 401 interchange improvements and grade separations at the major intersections. The location of these latter project sites are shown on the Key Plan, Figure 1.

2.1.2 SITE A - Essex County Road 19 and CPR Tracks

The existing Essex County Road 19 and Canadian Pacific Railroad intersection is an at grade crossing. The elevation of Essex County Road 19 at this location is approximately 182.3 metres.

The topography surrounding the crossing is generally flat to gently sloping towards the northeast with an average elevation of 181.0 metres.[1] The surrounding land use is primarily agricultural with some minor industrial/commercial properties. Site photographs are attached in Appendix D.

2.1.3 SITE B - Essex County Road 19 and Essex County Road 22

In the vicinity of this proposed grade separation, Essex County Road 19 is currently a two lane roadway carrying a single lane of traffic in each direction. Essex County Road 22 is a four lane divided roadway with two lanes in each direction. The interchange is bordered by residential properties to the northwest, commercial properties to the northeast, industrial/commercial properties to the southwest and vacant and/or agricultural properties to the southeast.

The existing elevation at the intersection is approximately 180 metres. The topography of the surrounding area is flat to gently undulating. Photographs of the site are attached in Appendix D.

2.1.4 SITE C - Essex County Road 22 and Lesperance Road

The existing Essex County Road 22 and Lesperance Road intersection is a crossing at grade. The elevation of Essex County Road 22 at this location is approximately 181.4 metres.

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[1] Surveys and Mapping Branch, Department of Energy, Mines and Resources. *Essex County, Ontario* [map] Edition 2. 1:25, 000. Ottawa, Ontario: Department of Energy, Mines and Resources, 1975.

The topography surrounding the intersection is generally flat to gently sloping towards the northeast with an average elevation of approximately 181.4 metres. The surrounding land use is primarily residential and commercial with some minor agriculture southeast of the intersection. Site photographs are attached in Appendix D.

2.2 Regional Geology

All three sites are located in the Essex Clay Plain, a subregion of the physiographic region of southern Ontario known as the St. Clair Clay Plain, as identified in "The Physiography of Southern Ontario", by Chapman and Putnam (1984). The clay plain is described as a till plain that has been locally smoothed by shallow deposits which settled in depressions in the till. The prevailing soil type is reported to be the Brookston clay.

Based on the Ontario Department of Mines and Northern Affairs Preliminary Map P.749 entitled "Quaternary Geology of the Windsor-Essex Area" (Western Part), the Lesperance Road and Essex County Road 22 intersection is located on the margin between predominantly clayey silt till and discontinuous, glaciolacustrine medium sand. The Manning Road sites at the CPR tracks and Essex County Road 22 are reportedly in areas characterized by clayey silt till.

The subcropping bedrock is reported to be limestone of the Dundee formation of Middle Devonian Age (Geological Survey of Canada, Map 1263A entitled "Geology, Toronto-Windsor Area", dated 1969). Based on the available information, the overburden thicknesses in the vicinity of the sites vary from 38 to 40 metres[2].

[2] Vagners, U.J, Sado, E.V., Yundt, S.E. *Drift Thickness Series Essex Area (Western Part)*. [map]. 1:50, 000, Preliminary Map 814. Ontario Department of Mines and Northern Affairs,1973.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between October 17 and November 5, 2007. Borehole 101 was advanced at Site A, the Essex County Road 19 – CPR crossing, between October 17 and October 22. Borehole 102 was drilled from October 23 to October 26 at Site B - the Essex County Road 19/Essex County Road 22 intersection. Borehole 103 was drilled from November 1 to November 5, 2007 at the Essex County Road 22/Lesperance Road intersection, Site C.

The investigation was carried out using truck mounted CME-55 and CME-75 drill rigs, equipped for auger boring and mud rotary drilling, which were supplied and operated by a specialist drilling contractor. Samples of the overburden were obtained at approximately 0.76 metre intervals to a depth of 5.0 metres (borehole 102), 6.2 metres (borehole 101) and 8.1 metres (borehole 103). Below these depths, samples were generally obtained at 1.5 metres intervals using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures. In the softer cohesive deposits, thin walled tube samplers were hydraulically advanced to procure relatively undisturbed samples. Groundwater conditions in the boreholes were observed throughout the drilling operations. Deep and shallow piezometers were installed in each of the borings upon completion of drilling. The boreholes were then backfilled in accordance with current MTO procedures and Ontario Regulation 128/03.

The field work was supervised on a full-time basis by experienced members of our engineering staff who located the boreholes in the field. The same individuals directed the drilling, sampling and in situ testing operations and logged the boreholes. The samples were identified at the site, placed in labelled containers and transported to our Mississauga and London laboratories for further examination and geotechnical laboratory testing. Routine index and classification tests consisting of water content determinations, grain size distribution analyses and Atterberg limits determinations were carried out on selected samples in our London facility and oedometer tests were performed on a single sample from each borehole in our Mississauga laboratory. The results of the testing are shown on the Record of Borehole sheets and in Appendices A through C.

The locations of the boreholes are shown on the Record of Borehole sheets and on Drawings 1 through 3 for sites A to C, respectively. The general borehole data is summarized in the following table.

SITE	BOREHOLE	LOCATION (m)		GROUND SURFACE ELEVATION (m)	BOREHOLE DEPTH (m)
		Northing	Easting		
A	101	4683241.7	274248.4	182.78	39.2
B	102	4685110.2	274370.2	179.22	35.1
C	103	4685326.7	273035.6	181.63	42.1

4.0 SUBSURFACE CONDITIONS

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the in situ testing and the laboratory testing carried out on selected samples, are given on the attached Record of Borehole sheets following the text of this report and in Appendices A through C. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous samples and observations of drilling resistance and, therefore, may represent transitions between soil types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

The locations and elevations of the boreholes are shown on the attached Drawings 1 through 3 together with stratigraphic profiles for the sites. Generalized geotechnical conditions are summarized in the sections below for the three project areas. For detailed descriptions of the subsurface conditions, reference should be made to the attached Record of Borehole sheets which take precedence.

4.1 Site Stratigraphy - SITE A - Essex County Road 19 and CPR Tracks

Borehole 101 encountered one metre of surficial fill and 0.3 metres of topsoil underlain by firm to hard cohesive till to a depth of 34 metres (approximately elevation 149 metres). The cohesive till contains layers of compact to very dense silty sand till/sandy silt till and is underlain by dense to very dense silt and silty sand till to a depth of 38 metres or elevation 144 metres. Borehole 101 was terminated at a depth of about 39 metres or at elevation 143 metres in assumed limestone bedrock.

4.1.1 Topsoil

Two layers of topsoil were encountered in borehole 101. The first layer was from surface for approximately 60 millimetres and the second layer was from elevation 181.7 metres for about 0.3 metres.

4.1.2 Fill

Fill materials were noted from approximately elevation 182.7 metres in borehole 101. The fill consisted of clayey silt and extended to approximately 1.1 metres depth. The fill material is firm with an N value of 6 blows per 0.3 metres.

4.1.3 Clayey Silt Till

An extensive stratum of clayey silt till was encountered in borehole 101 beneath the fill from elevation 181.4 metres. The clayey silt till contained two discrete layers of silty sand and sandy silt.

The upper portion of the clayey silt till was 8.6 metres thick and varied from firm to hard with N values of 7 to 70 blows per 0.3 metres. Four in situ vane shear strength tests were conducted in the softer zone of this material at elevations 175.3, 175.0, 173.8 and 173.5 metres. The in situ undrained shear strength of the clayey silt till ranged from 108 to greater than 143 kilopascals (increasing with depth) indicating a very stiff consistency.

The N values for the intermediate portion of the clayey silt till ranged from 7 to 23 blows per 0.3 metres characteristic of a firm to very stiff consistency. The lower portion of the clayey silt till ranged from firm to hard with N values of 7 to 37 blows per 0.3 metres. The clayey silt till samples had water contents ranging from 15 to 25 per cent. The clayey silt till material is of low plasticity based on the results of four Atterberg limits determinations. The Atterberg limits test results indicate average plastic and liquid limits of 15 and 32 per cent, respectively, and an average plasticity index of 17 per cent. The results of the Atterberg limits testing are shown on Figure A-4.

Grain size distribution curves for representative samples of clayey silt till recovered from the standard penetration testing are shown on Figure A-1.

A consolidation oedometer test was conducted on a sample of clayey silt till obtained from elevation 173.2 metres. The results of the laboratory consolidation testing are shown on Figures A-5 and A-6. The results indicate that the clayey silt till is over consolidated about 260 kilopascals beyond the existing overburden pressure. The following table summarizes the oedometer test results.

BOREHOLE AND SAMPLE	DEPTH (m)	σ'_p (kPa)	σ'_{vo} (kPa)	OCR	e_o	C_r	C_c	C_v (cm²/sec)
BH 101 SA 11	9.60	420	160	2.7	0.51	0.01	0.23	0.017

Although cobbles and boulders were not specifically encountered in the borehole advanced in this area, the presence of these materials should be expected due to the depositional environment of the glacial tills.

4.1.4 Silty Sand Till

Two layers of silty sand till were encountered in borehole 101, the first from elevation 172.8 metres within the clayey silt and the second from elevation 146.2 metres immediately overlying the bedrock. Grain size distribution curves for two samples of silty sand till recovered from the standard penetration testing are shown on Figure A-2. The silty sand till layers were 3.6 and 1.7 metres thick respectively.

The layers of silty sand till were very dense with N values of greater than 100 blows per 0.3 metres and natural water contents of 8 per cent for both samples. The silty sand till is marginally plastic based on average plastic and liquid limits of 11 and 13 per cent, respectively, and an average plasticity index of 2 per cent. The results of the Atterberg limits testing are shown on Figure A-4.

Although the boreholes did not specifically encounter cobbles and boulders in the silty sand till, given the variable nature of till deposits, their presence should be anticipated.

4.1.5 Sandy Silt Till

A 1.7 metre thick layer of sandy silt till was encountered in the clayey silt till from elevation 157.0 metres. The sandy silt till was compact with an N value of 13 blows per 0.3 m and a water content of 13 per cent.

4.1.6 Silt

A silt layer was encountered below the clayey silt till from elevation 149.2 metres. The silt is dense to very dense with N values of 49 to greater than 100 blows per 0.3 metres. The water content of the silt was 18 per cent and the material was found to be non-plastic. A grain size distribution curve for a silt sample is shown on Figure A-3.

4.1.7 Bedrock

Bedrock was encountered at approximately 38 metres depth or at elevation 144.5 metres based on the drill penetration rate and resistance. According to the geological literature for the area and an examination of the rock chips recovered, the bedrock likely consists of limestone from the Dundee Formation.

4.2 Site Stratigraphy - SITE B - Essex County Road 19 and Essex County Road 22

Borehole 102 encountered surficial fill to a depth of 0.7 metres or elevation 178.5. The fill is underlain by a relatively thick deposit of firm to very stiff clayey silt till to 31.4 metres or elevation 147.8 metres. The clayey silt till contains a thin layer of compact silty sand till. A layer of dense silt was found below the clayey silt till to a depth of about 32.9 metres (elevation 146.3 metres). The silt is underlain by a layer of very dense silty sand till to a depth of 34.0 metres or elevation 145.2 metres. Borehole 102 was terminated in inferred limestone bedrock at a depth of 35.1 metres or elevation 144.1 metres.

4.2.1 Topsoil

An 80 millimetre thick layer of topsoil was encountered at the surface of borehole 102.

4.2.2 Fill

The topsoil was underlain by a clayey silt fill layer, from elevation 179.1 metres, that was approximately 0.6 metres thick.

4.2.3 Clayey Silt Till

Clayey silt till was found beneath the fill from elevation 178.5 metres.

The N values for the majority of the clayey silt till layer ranged from 4 to 21 blows per 0.3 metres. Undrained shear strengths in the softer zones varied from 86 kilopascals to 112 kilopascals, indicative of a stiff to very stiff consistency. The clayey silt till became hard below about elevation 149.0 metres with an N value of 47 blows per 0.3 metres.

Water contents in the clayey silt till ranged from 18 to 24 per cent. The clayey silt till is of low plasticity based on average plastic and liquid limits of 15 and 29 per cent, respectively, and an average plasticity index of 15 per cent. The results of the Atterberg limits testing conducted on four samples are shown on Figure B-4.

Grain size distribution curves for four samples of clayey silt till retrieved from the standard penetration testing are shown on Figure B-1 in Appendix B.

Although not specifically encountered in the borehole, the presence of cobbles and boulders in the till should be expected.

Consolidation oedometer testing was conducted on a sample of clayey silt till obtained from elevation 169.8 metres. The results of the laboratory consolidation testing are shown on Figures B-5 and B-6. The results indicate that the clayey silt till is over consolidated about 115 kilopascals beyond the existing overburden pressure. The following table summarizes the oedometer test results.

BOREHOLE AND SAMPLE	DEPTH (m)	σ'_p (kPa)	σ'_{vo} (kPa)	OCR	e_o	C_r	C_c	C_v (cm^2/sec)
BH 102 SA 8	9.40	230	115	2.0	0.66	0.01	0.22	0.025

4.2.4 Silty Sand Till

Two layers of compact to very dense silty sand till were encountered at elevation 154.6 metres within the clayey silt till and at elevation 146.3 metres immediately below the silt.

The silty sand till had an N value of 74 blows per 0.3 metres and a water content of 7 per cent in the lower layer. The material has slight cohesion and is of low plasticity based on plastic and liquid limits of 11 and 13 per cent, respectively, and a plasticity index of 2 per cent. The results of the Atterberg limits testing conducted on a single sample are shown on Figure B-4.

The odour of hydrogen sulphide was noted in the sample of silty sand till retrieved during standard penetration testing.

A grain size distribution for a sample of the silty sand till recovered from the standard penetration testing is shown on Figure B-3.

Although cobbles and boulders were not specifically encountered in the boreholes advanced in this layer, the presence of these materials should be expected due to the nature of glacial tills.

4.2.5 Silt

The clayey silt till stratum was underlain by silt from approximately elevation 147.8 metres.

The silt was found to be dense with an N value of 49 blows per 0.3 metres and the water content for this material was 20 per cent. Based on the laboratory results, the silt has some slight cohesion and is of low plasticity based on plastic and liquid limits of 19 and 21 per cent, respectively, and a plasticity index of 2 per cent. The results of the Atterberg limits testing on a sample of the silt are shown on Figure B-4.

A grain size distribution curve for a sample of silt is shown on Figure B-2.

4.2.6 Bedrock

The bedrock surface was encountered at an approximate depth of 34.0 metres or at elevation 145.2 metres. Based on the fragments retrieved from the tricone drilling process, the bedrock was identified as limestone from the Dundee Formation.

4.3 Site Stratigraphy - SITE C - Essex County Road 22 and Lesperance Road

Borehole 103 intersected surficial fill and topsoil to a depth of 1.4 metres or elevation 180.3 metres and was underlain by a stratum of clayey silt till to 39.0 metres or approximately elevation 142.6 metres. The cohesive till contained four relatively thin layers of compact to very dense sandy silt till, silty sand till and silt ranging from 0.4 metres to 2.5 metres in thickness. Beneath the clayey silt till, the borehole encountered a layer of very dense silty sand till to a depth of 41.0 metres or elevation 140.6 metres. The borehole was terminated in presumed bedrock at an approximate depth of 42.1 metres (elevation of 139.6 metres).

4.3.1 Topsoil

Topsoil was noted from surface to approximately 150 millimetres depth in borehole 103.

4.3.2 Fill

Fill materials were encountered below the topsoil in borehole 103. The fill layers were 1.2 metres thick and consisted of clayey silt and crushed sand and gravel. The clayey silt fill was stiff with an N value of 11 blows per 0.3 metres.

4.3.3 Clayey Silt Till

An extensive stratum of clayey silt till was encountered below the fill in borehole 103.

The upper portion of the clayey silt till in borehole 103 was stiff to hard with N values of 10 to 37 blows per 0.3 metres above elevation 172.2 metres. In situ vane shear strength testing was conducted at elevations 173.1 and 172.8 metres in the clayey silt till. The undrained shear strengths ranged from 136 to greater than 144 kilopascals indicating a very stiff consistency. The middle portion of the clayey silt till layer, extending from elevation 169.1 metres to 155.4 metres, was firm to stiff with N values ranging from 5 to 8 blows per 0.3 metres. The lower zone of the clayey silt till, below elevation 155.0 metres, had N values that ranged from 7 to 40 blows per 0.3

metres indicating a firm to hard consistency. The bottom portion of the clayey silt till was hard with an N value of 54 blows per 0.3 metres.

The clayey silt till samples were found to have water contents ranging from 13 to 24 per cent.

Based on the results of six Atterberg limits determinations, the material is of low plasticity with average plastic and liquid limits of 14 and 29 per cent, respectively, and an average plasticity index of 15 per cent. The results of the Atterberg limits testing are shown on Figure C-3.

A consolidation oedometer test was conducted on a sample of clayey silt till obtained from elevation 172.2 metres. The results of the laboratory consolidation testing are shown on Figures C-4 and C-5. The results indicate that the clayey silt till is over consolidated about 225 kilopascals beyond the overburden pressure. The following table summarizes the oedometer test results.

BOREHOLE AND SAMPLE	DEPTH (m)	σ'_p (kPa)	σ'_{vo} (kPa)	OCR	e_o	C_r	C_c	C_v (cm²/sec)
BH 103 SA 11	9.4	350	125	2.8	0.45	0.01	0.23	0.0125

Grain size distribution curves from samples of the clayey silt till recovered from the standard penetration testing are shown on Figure C-1 in Appendix C.

Although cobbles and boulders were not specifically encountered in the borehole, their presence should be anticipated.

4.3.4 Sandy Silt Till

Layers of sandy silt till were encountered at 9.4 metres depth or elevation 172.2 metres and at 10.1 metres depth or elevation 171.6 metres. The layers were found to be 0.4 metres and 2.5 metres thick, respectively. The lower layer of sandy silt till was compact to very dense with N values ranging from 19 to 54 blows per 0.3 metres. A water content of 10 per cent was measured in a single sandy silt till sample. A grain size distribution curve for a sample of the sandy silt till recovered from the standard penetration testing is shown on Figure C-2.

4.3.5 Silty Sand Till

In borehole 103, two layers of silty sand till were encountered. The upper layer extended from elevation 155.4 metres for 0.6 metres depth and the lower layer extended below 142.6 metres for

2.0 metres. An N value of greater than 100 blows per 0.3 metres was measured in the deeper layer of silty sand till.

Cobbles and/or boulders were not specifically encountered in the till layers in the borehole; however, their presence should be expected.

4.3.6 Silt

A 1.5 metre thick layer of silt was encountered from elevation 145.7 metres in borehole 103. The silt was dense with an N value of 49 blows per 0.3 metres.

4.3.7 Bedrock

Bedrock was encountered at approximately 41.0 metres depth or at elevation 140.6 metres based on the drill penetration resistance and rate. Given the project location, the available geological literature and examination of the cuttings from the tricone drilling, the bedrock in this area likely consists of limestone from the Dundee Formation.

4.4 Groundwater Conditions

Groundwater conditions were observed during and on the completion of drilling and sampling. During drilling, groundwater was encountered at elevation 149.2 metres in borehole 101 and at elevation 145.7 metres in borehole 103. Both boreholes encountered groundwater at the interface of the clayey silt till and the underlying silt deposits. Borehole 102 was dry during drilling. Shallow and deep piezometers were installed in each of the boreholes upon termination of drilling and prior to backfilling. Based on the groundwater measurements in the shallow and deep piezometers, upward vertical hydraulic gradients were encountered in boreholes 102 and 103 on November 20, 2007 with an upward vertical gradient observed at borehole 102 only on February 14, 2008. Details of the groundwater conditions encountered and subsequently measured in the piezometers are provided on the Record of Borehole sheets. Groundwater data is summarized in the table below.

BOREHOLE	GROUND SURFACE ELEVATION (m)	ENCOUNTERED GROUNDWATER LEVEL		INSTALLATION	MEASURED GROUNDWATER LEVEL					
					Oct. 30 2007		Nov. 20, 2007		Feb. 14, 2008	
		Depth (m)	Elevation (m)		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
101	182.78	33.53	149.25	Shallow Piezometer	2.89	179.89	2.68	180.10	2.19	180.59
				Deep Piezometer	8.79	173.99	3.38	179.40	3.20	179.58

BOREHOLE	GROUND SURFACE ELEVATION (m)	ENCOUNTERED GROUNDWATER LEVEL		INSTALLATION	MEASURED GROUNDWATER LEVEL					
					Oct. 30 2007		Nov. 20, 2007		Feb. 14, 2008	
		Depth (m)	Elevation (m)		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
102	179.22	Dry	Dry	Shallow Piezometer	1.13	178.09	1.76	177.82	0.88	178.34
				Deep Piezometer	16.61	162.61	1.01	178.21	0.09*	179.13*
103	181.63	35.97	145.66	Deep Piezometer	N/A	N/A	2.57	179.06	2.50	179.13
103A	181.63	-	-	Shallow Piezometer	N/A	N/A	5.36	176.27	2.01	179.62

*Frozen on February 14, 2008.

Based on the drilling results at borehole 101, the long-term groundwater level is likely at approximately elevation 180 metres or about 3 metres below the existing ground surface. At borehole 102, the natural groundwater level is likely between 1 and 2 metres below surface or between elevation 177 and 178 metres. The long term groundwater level is inferred to be between elevation 176 and 179 metres or about 3 to 6 metres below the existing ground surface at the location of borehole 103.

The February 14, 2008 groundwater level measurements in boreholes 101 and 103 indicate a small (less than 1 metres) downward gradient. In borehole 102, the data indicates a small (less than 1 metre) upward gradient in borehole 102 on February 14, 2008 and a modest (2.8 metre) upward gradient on November 20, 2007.

The groundwater levels are expected to fluctuate seasonally and may be higher during periods of sustained precipitation or during the spring melt. Construction activity may also influence groundwater conditions.

5.0 MISCELLANEOUS

The investigation was carried out using drilling equipment supplied and operated by Aardvark Drilling Inc., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Mike Arthur under the direction of Mr. David J. Mitchell. The routine laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates.

The oedometer testing was carried out in Golder Associates' Mississauga laboratory under the direction of Dr. P. Dittrich, P.Eng. In addition to also being a participant in the MTO Soil and Aggregate Proficiency Program, the Mississauga laboratory is an MTO registered laboratory in the Specialty of Soil and Rock Including Testing for Foundation Engineering - Low and High Complexity.

This report was prepared by Miss Dirka U. Prout, P.Eng. under the direction of the Project Manager, Mr. Philip R. Bedell, P. Eng. This report was reviewed by Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

GOLDER ASSOCIATES LTD.

Dirka U. Prout, P.Eng.

Philip R. Bedell, P. Eng.
Principal

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

SG/DUP/PRB/FJH/cr
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PART B – PRELIMINARY FOUNDATION DESIGN REPORT

ESSEX COUNTY ROAD 19 AND CPR, ESSEX COUNTY ROAD 19 AND ESSEX COUNTY ROAD 22 AND ESSEX COUNTY ROAD 22 AND LESPERANCE ROAD GRADE SEPARATIONS
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
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MINISTRY OF TRANSPORTATION - SOUTHWESTERN REGION

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides our recommendations on the foundation aspects for the preliminary design of the proposed grade separations for the CPR crossing at Essex County Road 19, Site A (MTO Site B-19-04), the Essex County Road 19 intersection at Essex County Road 22, Site B (MTO Site B-22-03), and the Essex County Road 22 intersection at Lesperance Road, Site C (MTO Site 22-02), based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are preliminary and 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.

All three sites will feature high embankments with little opportunity for preloading.

It is understood that the existing at-grade intersection of the CPR tracks and Essex County Road 19 intersection will be replaced with a new CPCI concrete girder bridge with accompanying embankments. The proposed single span structure will be approximately 26 metres wide and 61 metres long with integral abutments founded on H-piles. The approximate proposed top of deck elevation is 191.7 metres. Retained Soil System (RSS) walls are to be constructed to retain embankment fills at the abutments. Based on the preliminary drawing, the embankments at this site will be approximately 9 metres high.

The Essex County Road 19/Essex County Road 22 Underpass Structure is to be a two span structure 36.6 metres long and 54 metres wide. The top of the structure will have a Single Point Urban Interchange (SPUI). The top of the deck elevation will vary between 185.7 to 186.2 metres. This structure will feature integral abutments supported by vertical steel H-piles. The central pier will be supported by battered steel H-piles. Embankment heights will be in the order of 6 metres. RSS walls will retain the approach fills for the ramps.

The future Essex County Road/Lesperance Road Overpass structure will be a single span structure 46 metres long and 36 metres wide. The integral abutments will be supported on steel H-piles with the approach fills retained with RSS walls. The top of deck elevation is about 188.0 metres. Embankment heights in the order of 7 metres have been proposed.

6.2 Bridge Foundations

6.2.1 SITE A - CPR and Essex County Road 19 Overhead (MTO Site B-19-04)

Borehole 101 encountered 1.1 metres of surficial fill and 0.3 m of topsoil underlain by firm to hard clayey silt till to a depth of 33.5 metres (elevation 149.2 metres). The clayey silt till contains layers of compact to very dense silty sand till/sandy silt till and is underlain by dense to very dense silt and silty sand till to a depth of 38.3 metres or elevation 144.5 metres. Borehole 101 terminated at a depth of about 39.2 metres or elevation 143.6 metres in assumed limestone bedrock.

Based on the preliminary information, the proposed CPR overhead structure will incorporate integral abutments. In this case, the abutments should be supported on steel H-piles driven to refusal in the upper very dense silty sand till, the lower very dense sandy silt till or on bedrock. If alternative structure types are considered, such as semi-integral abutments, consideration could be given to founding the abutments on closed-end 0.3 metre diameter steel tube piles bearing on the upper zone of the silty sand till (about elevation 171 metres) or perching the abutments on Granular A pads in the approach fills founded on the clayey silt till. However, due to the significant depth of moderately compressible soils in the area, the proposed high embankment fills and the limited opportunity for preloading, piles are the preferred founding option for the structure. The various foundation alternatives considered for Site A are compared in Table I. This table includes relative foundation costs and summaries of the feasibility of each option.

6.2.2 SITE B - Essex County Road 19 and Essex County Road 22 Underpass (MTO Site B-22-03)

Borehole 102 encountered subsurface conditions consisting of fill to an approximate depth of 0.7 metres or elevation 178.5 metres. The fill is underlain by a thick deposit of firm to very stiff clayey silt till to 31.4 metres or elevation 147.8 metres. The clayey silt till contains a thin layer of compact silty sand till at elevation 154.6 metres. A layer of dense silt was intersected below the clayey silt till to a depth of about 32.9 metres (elevation 146.3 metres). The silt is underlain by a layer of very dense silty sand till to a depth of 33.4 metres or elevation 145.2 metres. The borehole terminated in inferred limestone bedrock at a depth of 35.1 metres or elevation 144.1 metres.

Since integral abutments have been considered, the abutments should be supported on steel H-piles driven to refusal onto bedrock. Alternatively, in cases where integral abutments are not used, the structures could be founded on Granular A pads in the approach fills overlying the clayey silt till. As previously mentioned, given the significant depth of moderately compressible soils in the area, the proposed high embankment fills and the requirements for staged construction, piles are the preferred founding option for the structure. Comments on preliminary foundation costs and options for Site B are outlined in Table II.

6.2.3 SITE C - Essex County Road 22 and Lesperance Road Overpass (MTO Site 22-02)

Borehole 103 encountered fill and topsoil to a depth of 1.4 metres (elevation 180.3 metres) underlain by predominantly clayey silt till to 39.0 metres or approximately elevation 142.6 metres. The cohesive till contains four layers of compact to very dense sandy silt till, silty sand till and dense silt ranging from 0.4 metres to 2.5 metres in thickness. Beneath the clayey silt till, the borehole encountered a layer of very dense silty sand till to a depth of 41.0 metres or 140.6 metres elevation. The borehole was terminated in presumed bedrock at an approximate depth of 42.1 metres (elevation of 139.5 metres).

Integral abutments should be supported on steel H-piles driven to refusal into the lower very dense sandy till or bedrock. If other structure types are considered, the abutments could be founded on Granular A pads in the approach fills overlying the clayey silt till. As outlined for the other two sites, piles are the preferred founding option for the structure. Comments on the foundation options and costs for the Essex County Road 22/Lesperance Road location (Site C) are outlined in Table III.

6.3 Deep Foundations

It is understood that consideration is being given to designing the structures at the grade separations with integral abutments at all three sites.

SITE A

For preliminary design purposes at the CPR/Essex County Road 19 site, the abutments at Site A can be supported on driven HP 310 x 110 steel H-piles advanced to refusal into the sandy silt till (elevation 145.0 metres) or bedrock (elevation 144.5 metres). It should be noted that approximately 6 to 8 metres of hard driving would be experienced in the very dense silty sand till deposit below elevation 173 metres and in the dense to very dense silt and silty sand till deposits below elevation 149 metres. Alternatively, abutment piles consisting of either HP 310 x 110 steel H-piles or closed-end 0.3 metre diameter steel tube piles (for non-integral abutments) could be founded in the very dense silty sand till with a tip elevation of approximately 171 metres at this location. It should be noted that this option will require a thorough investigation to confirm the thickness, extent and nature of the layers during detail design. Further, care will be required during pile driving to ensure that the piles are not overdriven into the lower clayey silt till deposit.

SITE B

Integral abutments at the Essex County Road 22/Essex County Road 19 site can be supported on HP 310 x 110 steel H-piles driven to refusal on bedrock at approximately elevation 145.2 metres. Based on the investigation results, approximately 4 metres of hard driving can be expected through the clayey silt till, silt and silty sand till immediately above the bedrock surface.

SITE C

At the Essex County Road 22/Lesperance Road location, abutments can also be supported on HP 310 x 110 steel H-piles driven to refusal in the silty sand till with a tip elevation of approximately 141.6 metres or on bedrock at about elevation 140.6 metres. The borehole results indicate two zones of potentially hard pile driving for a total of 7 metres. The upper difficult driving zone consists of a sandy silt till layer below elevation 171 metres and the lower zone is located below elevation 147 metres and extends to the till/bedrock interface.

Driven steel piles founded on bedrock are the preferred foundation system for the abutments at all three locations. Pre-augering and placement of a corrugated steel pipe (CSP) liner filled with loose sand around the upper 3 metres of the pile is required to reduce resistance to lateral movement. A Non Standard Special Provision (NSSP) for CSP Integral Abutments detailing the sand gradation should be included in the Contract Documents.

Geotechnical Axial Resistance – Driven Steel Piles

The following table summarizes the factored axial geotechnical resistances at Ultimate Limit States (ULS) and unfactored resistance at Serviceability Limit States (SLS) for the foundation options at the three sites.

SITE	FOUNDATION OPTION	FOUNDING STRATA			GEOTECHNICAL RESISTANCE	
		Description	Elevation (m)	Depth (m)	Factored ULS (kN)	SLS[1] (kN)
A	0.3 m dia closed end tube HP 310x110 HP 310x110 HP 310x110	Silty Sand Till Silty Sand Till Silty Sand Till Bedrock	172 172 145 144	11 11 37 39	1400 1200 1600 2000	1100 900 1400 -
B	HP 310x110	Bedrock	145	34	2000	-
C	HP 310x110 HP 310x110	Silty Sand Till Bedrock	142 140	40 42	1600 2000	1400 -

1. SLS values have not been provided for end bearing piles driven to refusal in the limestone bedrock because it is considered to be an unyielding medium.

The steel piles should be installed and monitored in accordance with SP903S01. Since hard driving conditions are expected at depth, H-piles should be equipped with driving shoes such as Titus 'H' Bearing Pile Point (Standard Model) or equivalent. Driving shoes and steel pile splices are to be installed in accordance with OPSD 3000.100 and OPSD 3000.150, respectively. Further, piles driven into the silty sand till deposits should be installed in accordance with Standard SS103-11 and the piles should be retapped as required by SP903S01 in order to confirm the set after adjacent piles have been driven.

In addition to the above, for the purposes of construction, an ultimate geotechnical resistance of 2 times the design load at ULS is to be specified and should be added to the notes on the drawings. In case of founding the steel tube or H-piles on the upper silty sand layer at Site A, care must be taken not to drive the pile through the very dense silty sand till layer. The notes on the CPR overhead drawings should specify that the piles must be driven to below elevation 172 metres and not below elevation 170 metres without the approval of the Engineer. For piles driven to bedrock, retapping is not necessary. The applicable pile note is "Piles to be driven to bedrock".

If deep foundations are selected at the detail design stage, the elevations of the founding layers as well as the depth to bedrock should be confirmed through additional borings at the abutment locations and the proposed piers (if considered) to confirm anticipated pile lengths and founding conditions.

As noted in Section 4.4, the current piezometer data indicates slight to moderate upward gradients in boreholes 102 (Site B) and borehole 103 (Site C), respectively. Based on the existing data which indicates that both the upper and lower groundwater levels are below ground surface, it is not anticipated that this phenomena will have an adverse impact on pile capacity. However, additional groundwater level measurements will be required during detailed design to confirm and/or refine the groundwater regime at these sites.

Downdrag Load (Negative Skin Friction)

The approach embankments will induce consolidation settlement of the underlying cohesive till deposits. The consolidation settlement is time-dependent and will not completely occur during the construction period unless the embankments are placed well in advance of bridge construction. Post-construction settlement of the cohesive deposits will take place and settlement of the clayey materials 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.

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 and the surface area of the pile within the clay deposit. The unit negative skin friction acting on a unit area along a single pile can be calculated using the following equation:

$$f_{ns} = \beta \, \sigma_v'$$

Where	f_{ns}	=	unit negative skin friction
	β	=	shaft resistance factor = 0.25
	σ_v'	=	Effective vertical (overburden) pressure

For these sites σ_v' Can be calculated (approximately) for design purposes as $\sigma_v' = \gamma' z$

Where	γ'	=	buoyant unit weight of soil (10 kN/m^3)
	z	=	depth below final road surface (m)

The total downdrag load is a function of the surface area of the pile within the cohesive soil. 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. Based on the results of the investigation, the unfactored downdrag load acting on the piles may be taken as 500 kilonewtons per pile when founded on the lower silty sand/silt or bedrock and as 250 kilonewtons per pile when set in the upper silty sand till at the CPR/Essex County Road 19 location. In order to minimize settlement induced by downdrag loads on the piles, the embankments could be constructed well in advance of installation of the piles.

Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. In the case of integral abutments, vertical piles must provide the resistance to the lateral loading and the horizontal reaction to the pile can be estimated using the following equation and the ranges in subgrade reaction coefficient provided for all three sites.

k_s = coefficient of horizontal subgrade reaction (MPa/m) $= n_h (z/d)$ for cohesionless soils

$= \dfrac{67\, S_u}{d}$ for cohesive soils

d = pile width or diameter (m)

n_h = constant of horizontal subgrade reaction (MPa/m)

S_u = undrained shear strength of the soil (MPa)

z = depth below ground surface grade (m)

SITE A - CPR Tracks and Essex County Road 19

SOIL TYPE	ELEVATION (m)		n_h	S_u
	From	To	(MPa/m)	(MPa)
Backfill around piles and CSPs (assumed to be granular fill)	Surface	180	5 to 10	-
Firm to hard clayey silt till	180 169	173 149	- -	0.050 to 0.320 0.050 to 0.250
Dense to very dense silt Very dense silty sand till	149 173 146	146 169 145	10 to 12 15 to 25 10 to 12	- - -

SITE B - Essex County Road 22 and Essex County Road 19

SOIL TYPE	ELEVATION (m)		n_h	S_u
	From	To	(MPa/m)	(MPa)
Backfill around piles and CSPs (assumed to be granular fill)	Surface	176	5 to 10	-
Firm to very stiff clayey silt till	176	148	-	0.050 to 0.200
Compact silty sand till	154	153	2 to 5	-
Dense silt Very dense silty sand till	148 146	146 145	9 to 11 10 to 12	- -

SITE C - Essex County Road 22 and Lesperance Road

SOIL TYPE	ELEVATION (m)		n_h	S_u
	From	To	(MPa/m)	(MPa)
Backfill around piles and CSPs (assumed to be granular fill)	Surface	179	5 to 10	-
Firm to hard clayey silt till Firm to stiff clayey silt till Hard clayey silt till	179 169 144	172 146 143	- - -	0.050 to 0.320 0.050 to 0.100 0.200 to 0.320
Compact to very dense sandy silt till Dense silt Very dense silty sand till	172 146 143	169 144 141	7 to 15 9 to 11 10 to 12	- - -

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*[1]
8d	1.00
6d	0.70
4d	0.40
3d	0.25

For the driven steel pile options, unfactored lateral resistances of 150 kN at ULS and 70 kN at SLS in the clayey silt till and compacted cohesive fill material, respectively can be used based on a lateral movement of 10 millimetres.

Frost Protection

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

6.4 Shallow Foundations

Shallow foundations such as spread footings are not suitable for integral abutments. However, at pier locations, or if non-integral abutments are to be constructed, the abutments and/or piers of the future structures could be founded on spread footings bearing on the surface of the very stiff to hard clayey silt till crust. Alternatively, the abutments could be founded on footings perched in the abutment fills. Shallow foundations are not the preferred founding option for the abutments due to the potential for settlement of the underlying moderately compressible cohesive soils induced by the embankment loading.

Geotechnical Resistance

Spread footings could be founded below all fill and topsoil layers on the very stiff to hard clayey silt till at or below the following elevations:

SITE	BOREHOLE	RECOMMENDED FOUNDATION ELEVATION (m)	GEOTECHNICAL RESISTANCE Factored ULS (kPa)	SLS (kPa)
A	101	179.5	450	300
B	102	177.0	250	175
C	103	179.5	375	250

[1] Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2. Department of the Navy, Naval Facilities Engineering Command (1982).

A maximum footing width of 3 metres has been assumed. The SLS value is based on 25 millimetres of settlement. The settlement(s) of these footings will be dependent on the actual footing size, configuration and applied loads.

Alternatively, perched abutments on compacted Granular A constructed within the approach embankment fills could be designed for a factored geotechnical resistance at ULS of 900 kilopascals and an SLS value of 350 kilopascals based on 25 millimetres of settlement. For either shallow foundation option, additional settlement of the abutment footings will occur due to consolidation of the founding soils caused by the embankment construction. Therefore, the embankments should be placed well in advance to reduce the impacts of the 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 CHBDC.

Further, the above geotechnical resistances assume that appropriate construction procedures are adopted during footing construction to ensure that the founding soils are not softened/disturbed prior to concrete placement and the bearing surface is dry. If shallow footings are considered, additional boreholes will be required to confirm the founding depth and geotechnical resistances for detail design.

Resistance to Lateral Forces

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

Footings on clayey silt till	angle of friction, δ,	28°
	tan δ	0.53

In accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the factored horizontal resistance.

Frost Protection

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

Construction Considerations

The founding soils are sensitive to disturbance and loosening due to water seepage and/or ponding. Placement of a working slab will be required at the base of excavation for the footing area. Exposure without protection of the working slab may result in disturbance of the bearing surface. The cleaned excavation base should be inspected by qualified geotechnical personnel prior to placing the working slab. It is recommended that the footing excavation be carried out such that the final 0.5 metres of excavation is completed with the geotechnical personnel on site and that the working slab be placed immediately after footing inspection.

6.5 Retained Soil Systems

It is understood that mechanically-reinforced soil retaining wall systems (Retained Soil System or RSS walls are proposed for use at the abutments for all three sites. According to the preliminary design drawings, the RSS walls will have high appearance and high performance. These reinforced earth fills will be up to 9 metres high. The areas of proposed filling should be prepared by removing all topsoil and deleterious materials and proofrolling the exposed subgrade to delineate soft spots and the like.

The use of RSS walls is considered appropriate for retaining walls or wing walls founded on the hard clayey silt till. The RSS walls should be founded at about elevation 180 metres (elevation 178 metres at Site B). The excavations for the wall facing should be backfilled with compacted Granular A or Granular B Type III to underside of footing elevation. Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which is generally taken as 0.7 times the height of the wall, and a footing width of 1 metre, a factored geotechnical resistance at ULS of 350 kilopascals may be used for design of RSS walls founded on a properly prepared sand and gravel subgrade at both the west and east abutments for Sites A and C and a factored geotechnical resistance at ULS of 250 kilopascals may be used for Site B at both abutments. The geotechnical resistance at SLS for 25 millimetres of settlement considering the footing loading only may be taken as 225 kilopascals for both abutments for Sites A and C. The geotechnical resistance at SLS for Site B may be taken as 175 kilopascals for 25 millimetres of settlement for both abutments. Additional settlements of the earth fill embankment will occur as discussed in Section 6.4.

The resistance to lateral forces/sliding resistance between the compacted granular fill (assumed to be Granular A) and the subgrade soils should be calculated in accordance with Section 6.7.5 of the CHBDC. The unfactored coefficient of friction, tan δ, may be taken as 0.6 between the compacted Granular A and the concrete footing. In accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the factored horizontal resistance.

The internal stability of the mechanically-reinforced soil walls should be checked by the RSS supplier/designer. The Factor of Safety related to global stability under static loading for properly designed and constructed RSS walls at this site is greater than 1.3.

The design and construction of the RSS walls should be carried out in accordance with the manufacturer's design recommendations and MTO Special Provisions SP599S22 and SP599S23.

6.6 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 Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B but with less than 5 per cent passing the 75 µm sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 millimetres in thickness in accordance with SP 105S10. 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 tapers should be in accordance with Ontario Provincial Standard Drawing (OPSD) 3101.150, 3190.100 and 3121.150.
- A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design of the abutment walls in accordance with CHBDC Figure 6.6. Compaction equipment should be used in accordance with SP 105S10.
- 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 a from Commentary on CHBDC Clause C6.9.1) 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 b from Commentary on CHBDC Clause C6.9.1).
- For Case a, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be assumed:

Soil unit weight:	21 kN/m³
Coefficients of lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50

- For Case b, 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

- Where lightweight slag fill is installed behind the abutment wall, the pressures acting over the depth of the lightweight slag fill may be calculated as follows:

slag fill unit weight 12 kN/m³

coefficients of static lateral earth pressure:

Active, K_a	0.27
At rest, K_o	0.43
Passive, K_p	3.7

- 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. For sloping backfill/ground surface, these parameters should be adjusted as indicated in CHBDC C6.9.1(e).

6.7 Embankments

It is understood that the embankments will be approximately 9 metres high at the CPR crossing with Essex County Road 19, 6 metres high at the Essex County Road 19/Essex County Road 22 intersection and 7 metres high at Essex County Road 22/Lesperance Road intersection. The fill materials are to consist of well compacted on site borrow materials free of organics. Embankment side slopes formed no steeper than 2 horizontal to 1 vertical are considered suitable for the three sites. A Factor of Safety against deep seated failure of greater than 1.3 is available for embankments constructed with native materials founded on the stiff to hard clayey silt till subgrade soils.

The topsoil, fill, organic and soft materials should be removed from within the embankment footprints and the exposed subgrade soils should be proofrolled prior to fill placement under the direction of qualified geotechnical personnel.

Construction of the embankments 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 and compacted in regular lifts with loose thickness not exceeding 300 millimetres. Also, any embankments in excess of 8 metres in height should be provided with a 2 metre wide bench at mid-height.

6.7.1 Settlement Analyses

Settlement analyses were carried out for the proposed approach embankments at the three sites based on the borehole data obtained during the investigation, consolidation tests and the preliminary design information and Geocres Report No. 40J2-79 entitled "Structure Settlement Study, Highway 401 Reconstruction, GWP 64-00-00, Ministry of Transportation, Ontario, Southwestern Region", dated November 22, 2006. The embankment fill loads were modelled as two approach wedges to the bridge structure with approach grades of 4 per cent in the case of the CPR/Manning Road location and as embankments with 4 per cent grades (from the east and the west) to the maximum height outlined above for the Essex County Road 22 sites. In all cases, the embankments were modelled using the proposed cross-sectional dimensions of the proposed fills. The embankment fills, loads and resulting settlements were modelled using Settle 3D using the following parameters:

SITE	σ'_p	σ'_{vo}	e_o	C_r	C_c
A	420	160	0.41	0.01	0.23
B	230	115	0.66	0.01	0.22
C	350	125	0.45	0.01	0.23

Based on the computer modelling, the magnitude of the settlements at each site could be reduced by approximately one half through the use of lightweight fill such as slag. A unit weight of 12 kilonewtons per cubic metre was assumed for the lightweight slag.

Site A - CPR Tracks and Essex County Road 19 Overhead

Given the results of the analyses, it is estimated that some 150 millimetres of total settlement of the completed standard fill embankments will occur at the maximum height. In the case that lightweight fill is used, it is estimated that approximately 80 millimetres of total settlement will occur at the maximum fill height. Furthermore, it is estimated that 90 per cent of this settlement will occur within about three years. It is recommended that the embankment construction be commenced as soon as possible in advance of bridge construction to allow some of this settlement to take place and reduce downdrag loads. If standard fill is used, bridge construction should commence no sooner than six months after placement of embankment fills or when there is less than 100 millimetres of settlement remaining.

The preliminary settlement analyses did assess the effect(s) of embankment construction and settlement on the Union Gas main leading to the sub-station at the southwest corner of the intersection of the CPR line and Essex County Road 19. For detailed design, the location of the gas main must be accurately documented and a refined settlement analysis conducted to examine the impact of embankment construction on the gas main.

Site B - Essex County Road 22 and Essex County Road 19 Underpass

At this location, it is estimated that approximately 120 millimetres of total settlement will occur under an embankment constructed with standard fill at the maximum height and approximately 60 millimetres will occur under a lightweight fill embankment at maximum height. In this case, approximately 90 per cent of the settlement will occur after about three years. It is recommended that the embankment be placed as soon as possible in advance of bridge construction.

In all cases, downdrag loads on the bridge piles will be reduced if they are installed after 100 millimetres or less of settlement, due to the embankment fills, is remaining. This corresponds to a recommended minimum wait time of four months to begin bridge construction if standard fill is used.

Site C - Essex County Road 22 and Lesperance Road Overpass

Approximately 110 millimetres of total settlement is expected under the maximum height of the standard fill embankment at this location. It is expected that approximately 60 millimetres of total settlement will occur under an embankment constructed with lightweight fill at the maximum height. Some 90 per cent of the settlement will take place in approximately three years. Again, it is recommended that the embankment be constructed at the earliest possible date in order to facilitate settlement prior to the building the overpass structure. If standard fill is sued for embankment construction, downdrag loads on the foundation piles for the structure will be minimized if construction begins after 100 millimetres or less of settlement is remaining. This corresponds to a minimum time of two months between the completion of embankment construction and the start of bridge construction.

6.8 Excavations and Temporary Cut Slopes

Excavations for the pile caps and shallow spread footings (if utilized) will penetrate the existing fill into the clayey silt till. The excavations will likely encounter groundwater; however, seepage volumes are expected to be low due to the fine grained nature of the soil. Given the available geotechnical information, groundwater can be controlled by using properly filtered sumps.

The founding soils should be protected as noted in Section 6.2.2 under the heading Construction Considerations. 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 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 fill materials at this site would be classified as Type 3 soils as would any cohesionless materials below the groundwater level. The native clayey materials and properly dewatered cohesionless materials would be classified as Type 1 or Type 2 depending on consistency or relative density.

Staged construction has been proposed for the Essex County Road 22/Lesperance Road overpass site. Temporary traffic protection will be required from Stage S1 when the southern half of the overpass structure will be erected and Lesperance Road lowered. At Stage S2, Essex County Road 22 traffic will be switched to the new structure while the northern half is erected and the northern section of Lesperance Road regraded. Temporary roadway protection (shoring) will have to be provided to support the embankment fills and excavations in restricted areas during construction in accordance with SP105S19. The shoring should be designed to Performance Level 2.

7.0 MISCELLANEOUS

This report was prepared by Miss Dirka U. Prout, P.Eng. under the direction of the Project Manager, Mr. Philip R. Bedell, P. Eng. and the report was reviewed by Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

GOLDER ASSOCIATES LTD.

Dirka U. Prout, P. Eng.

Philip R. Bedell, P. Eng.
Principal

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

SG/DUP/PRB/FJH/cr
n:\active\2006\1130 - geotechnical\1130-100\06-1130-180 dillon - manning road fdns - essex\reports\-3) cpr - lesperance - hwy 22 - grade separation\feb 15 08 - (final) parts a and b - cr 22 - cpr - lesperance rd grade separations.doc

TABLE I

COMPARISON OF FOUNDATION ALTERNATIVES

Essex County Road 19 and CPR Crossing Grade Separation
Essex County Road 19 (Manning Road) Widening
GWP 3031-06-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
End bearing steel H-pile foundations driven to practical refusal in the lower silty sand till or on bedrock	• Feasible for support of all foundation elements	• High bearing resistance • Negligible settlement	• More costly than shallow footings and short piles • Use of piled foundations results in construction schedule of longer duration	• More expensive than shallow foundations and short piles	• Pile driving may be adversely affected by obstructions (cobbles and boulders) and hard driving in the very dense silty sand below elevation 173 metres and clayey silt till, silt and silty sand till below elevation 151 metres • Embankment construction will induce pile settlement due to downdrag load.
Spread footings supported either directly on crust of clayey silt till or perched on granular pad in abutment fills	• Not compatible with proposed integral abutments • Feasible for semi-integral abutments	• Least Expensive option • Ease of construction	• Time and cost of settlement mitigation measures; if time is available, preloading, possibly with surcharging, would be relatively inexpensive • Even if mitigation adopted, settlement of shallow foundations could still take place • Large differential settlements may occur due to larger settlements at abutment locations due to embankment fill.	• Excluding cost of mitigation measures, costs are expected to be less expensive than deep foundation options	• Even if mitigation in place, shallow foundations may still be affected by settlement of underlying cohesive deposits

COMPARISON OF FOUNDATION ALTERNATIVES

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
End bearing steel H-pile or closed-end 0.3 metre diameter tube pile driven to practical refusal in the upper silty sand till	• Tube piles not compatible with proposed integral abutments • H-piles feasible for all foundation elements	• Relatively high bearing resistance	• More costly than shallow footings • Use of piled foundations result in construction schedule of longer duration • Larger settlements than deep pile option due to consolidation of thick cohesive deposit below founding layer • Care must be taken during driving to ensure that piles do not penetrate upper silty sand till • Requires a more comprehensive geotechnical investigation to determine lateral extents and properties of the proposed founding layer	• Excluding cost of mitigation measures, more expense than shallow foundation, less expensive than deep pile option	• Even if mitigation in place, piles may still be affected by settlement of underlying cohesive deposits • Pile driving may be adversely affected by obstructions (cobbles and boulders) and thin founding strata. • Embankment construction will induce pile settlement due to downdrag load.

NOTES:

1. Quantitative cost estimates will be provided when more detailed design information is available.
2. Table to be read in conjunction with accompanying report.

Prepared By: SG

Checked By: PRB

TABLE II

COMPARISON OF FOUNDATION ALTERNATIVES

Essex County Road 19 and Essex County Road 22 Grade Separation
Essex County Road 19 (Manning Road) Widening
GWP 3031-06-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
End bearing steel H-pile foundations driven to practical refusal on bedrock	• Feasible for support of all foundation elements	• High bearing resistance • Negligible settlement	• More costly than shallow footings • Use of piled foundations results in construction schedule of longer duration	• More expensive than shallow foundations	• Pile driving may be adversely affected by obstructions (cobbles and boulders) and hard driving in the hard silty clay till, dense silt and very dense silty sand till below elevation 149 metres • Embankment construction will induce settlement of the piles due to downdrag load.
Spread footings supported either directly on crust of clayey silt till or perched on granular pad in abutment fills	• Not compatible with proposed integral abutments • Feasible for semi-integral abutments	• Least Expensive option • Ease of construction	• Time and cost of settlement mitigation measures; if time is available, preloading, possibly with surcharging, would be relatively inexpensive • Even if mitigation adopted, settlement of shallow foundations could still take place • If design features integral abutments, spread footing only suitable for piers • Large differential settlements may occur due to larger settlements at abutment locations due to embankment fill.	• Excluding cost of mitigation measures, costs are expected to be less expensive than deep foundation options	• Even if mitigation in place, shallow foundations may still be affected by settlement of underlying cohesive deposits

NOTES:
1. Quantitative cost estimates will be provided when more detailed design information is available.
2. Table to be read in conjunction with accompanying report.

Prepared By: SG

Checked By: PRB

TABLE III

COMPARISON OF FOUNDATION ALTERNATIVES

Essex County Road 22 and Lesperance Road Grade Separation
Essex County Road 19 (Manning Road) Widening
GWP 3031-06-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
End bearing steel H-pile foundations driven to practical refusal in the lower silty sand till or bedrock	• Feasible for support of all foundation elements	• High bearing resistance • Negligible settlement	• More costly than shallow footings • Use of piled foundations results in construction schedule of longer duration	• More expensive than shallow foundations	• Pile driving may be adversely affected by obstructions and hard driving in the upper very dense sandy silt till below elevation 171 metres as well as lower hard silty clay till, dense silt and very dense silty sand till below elevation 147 metres • Embankment placement will induce settlement of the piles due to downdrag load.
Spread footings supported either directly on crust of clayey silt till or perched on granular pad in abutment fills	• Not compatible with proposed integral abutments • Feasible for semi-integral abutments	• Least Expensive option • Ease of construction	• Time and cost of settlement mitigation measures; if time is available, preloading, possibly with surcharging, would be relatively inexpensive • Even if mitigation adopted, settlement of shallow foundations could still take place • If design features integral abutments, spread footing only suitable for piers • Large differential settlements may occur due to larger settlements at abutment locations due to embankment fill.	• Excluding cost of mitigation measures, costs are expected to be less expensive than deep foundation options	• Even if mitigation in place, shallow foundations may still be affected by settlement of underlying cohesive deposits

NOTES:
1. Quantitative cost estimates will be provided when more detailed design information is available.
2. Table to be read in conjunction with accompanying report.

Prepared By: SG

Checked By: PRB

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

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.)

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, 60º 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 cm^2 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.

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

(b) Cohesive Soils

Consistency

	c_u, s_u kPa	c_u, s_u psf
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

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
v	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 1
w_l	liquid limit
w_p	plastic limit
l_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 = σ'_p/σ'_{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 δ
c'	effective cohesion
c_u, s_u	undrained shear strength (ϕ = 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)

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 101

METRIC

PROJECT 06-1130-180-0-3
G.W.P. 3031-06-00
LOCATION N 4683241.7 ;E 274248.4
ORIGINATED BY MA
DIST HWY
BOREHOLE TYPE POWER AUGER/HOLLOW STEM & MUD ROTARY
COMPILED BY BRS
DATUM GEODETIC
DATE October 17, 2007 - October 18, 2007
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	Water content / shear strength	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.78	GROUND SURFACE						
0.06	TOPSOIL, sandy Brown						
	(FILL), clayey silt, trace sand, trace gravel Firm Brown	1	SS	6			
181.68 1.10	TOPSOIL, clayey Firm Brown						
1.37	CLAYEY SILT (TILL), some sand, trace gravel Firm to hard Brown becoming grey below about elev. 178.0m	2	SS	9			
		3	SS	7			
		4	SS	70			
		5	SS	44			2 22 44 32
		6	SS	40			
		7	SS	18			
		8	SS	13			
		9	SS	8			
					Field vane: 2.0; 1.6		
		10	SS	10			
					1.5; 144; >144		
172.84		11	SS	PH		21.0	5 39 40 16 Consolidation
9.94	SILTY SAND (TILL), some clay, trace gravel Very dense Grey						
		12	SS	100			8 48 33 11
		13	SS	100			
169.22 13.56	CLAYEY SILT (TILL), trace sand, trace gravel Firm to very stiff Grey						
		14	SS	12			

Elevation scale: 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168

Bentonite; Sand Fill; Grout

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

Continued Next Page

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

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London, Ontario

RECORD OF BOREHOLE No 101

METRIC

PROJECT 06-1130-180-0-3

G.W.P. 3031-06-00

LOCATION N 4683241.7 ;E 274248.4

ORIGINATED BY MA

DIST HWY

BOREHOLE TYPE POWER AUGER/HOLLOW STEM & MUD ROTARY

COMPILED BY BRS

DATUM GEODETIC

DATE October 17, 2007 - October 18, 2007

CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		15	SS	23	Grout	167		
						166		
		16	SS	12		165		
						164		
		17	SS	PH		163		
		18	SS	8		162	w_P – w – w_L	2 18 38 42
		19	SS	7		161	w	
		20	SS	9		160 / 159		
		21	SS	10		158		
157.02 / 25.76	SANDY SILT (TILL), trace clay, trace gravel Compact Grey					157		
		22	SS	13		156	w	
155.35 / 27.43	CLAYEY SILT (TILL), some sand, trace gravel Firm to hard Grey					155		
		23	SS	7		154	w_P – w – w_L	2 20 40 38
		24	SS	10		153	w	

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

Legend: 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/m^3

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RECORD OF BOREHOLE No 101

METRIC

PROJECT 06-1130-180-0-3
G.W.P. 3031-06-00
LOCATION N 4683241.7 ;E 274248.4
ORIGINATED BY MA
DIST HWY
BOREHOLE TYPE POWER AUGER/HOLLOW STEM & MUD ROTARY
COMPILED BY BRS
DATUM GEODETIC
DATE October 17, 2007 - October 18, 2007
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
					Grout	152						
		25	SS	8		151						
		26	SS	37		150						
149.25 33.53	SILT, some clay, trace sand, trace gravel Dense to very dense Grey				∇	149						
		27	SS	104/ 225mm		148	○		3	1	85	11
		28	SS	49		147						
146.20 36.58	SILTY SAND (TILL), trace gravel Very dense Grey					146						
		29	SS	108	Cuttings	145	○ H		5	47	35	13
144.48 38.30	LIMESTONE BEDROCK (inferred) Grey (Dundee Formation)	~~30~~	~~SS~~	100/ 0mm		144						
143.58 39.20	END OF BOREHOLE											

Groundwater encountered at about elev. 149.25m during drilling on October 18, 2007.

Groundwater measured in shallow piezometer at elev. 180.10m on November 20, 2007.

Groundwater measured in deep piezometer at elev. 179.40m on November 20, 2007.

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
REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

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RECORD OF BOREHOLE No 102

METRIC

PROJECT 06-1130-180-0-3
G.W.P. 3031-06-00
LOCATION N 4685110.2 ;E 274370.2
ORIGINATED BY MA/BG
DIST HWY
BOREHOLE TYPE POWER AUGER / HOLLOW STEM & MUD ROTARY
COMPILED BY BRS
DATUM GEODETIC
DATE October 23, 2007 - October 26, 2007
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	SHEAR STRENGTH kPa (FIELD VANE, sensitivity)	WATER CONTENT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.22	GROUND SURFACE								
0.08	TOPSOIL, silty; Brown								
178.52 / 0.70	(FILL), clayey silt, trace sand, trace gravel, trace to some topsoil; Brown								
	CLAYEY SILT (TILL), some sand, trace gravel; Firm to very stiff; Brown becoming grey at about elev. 175.0m	1	SS	9	Bentonite				
		2	SS	7					
		3	SS	14			✓		1 20 44 35
		4	SS	16	Sand Fill				
		5	SS	21					
		6	SS	6			✓		0 23 45 32
						+ 4.0; + 2.4			
		7	SS	5	Bentonite				
						+ 5.0; + 1.7			
					Grout				
		8	TO	PH			✓	20.0	1 19 48 32 Consolidation
						+ 1.7; + 1.2			
		9	SS	5					
		10	SS	6			○		
		11	SS	4					

Elevation scale: 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165

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

Continued Next Page

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

LDN_MTO_01 06-1130-180-0-3.GPJ LDN_MTO.GDT 2/11/08

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RECORD OF BOREHOLE No 102

METRIC

PROJECT 06-1130-180-0-3
G.W.P. 3031-06-00
LOCATION N 4685110.2 ;E 274370.2
ORIGINATED BY MA/BG
DIST HWY
BOREHOLE TYPE POWER AUGER / HOLLOW STEM & MUD ROTARY
COMPILED BY BRS
DATUM GEODETIC
DATE October 23, 2007 - October 26, 2007
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
					164		
		12	TO	PH	Grout		
					163		
		13	SS	5	162	o	
		14	SS	7	161		
					160		
		15	SS	6	159	⊢o⊣	2 21 41 36
					158		
		16	SS	8	157		
		17	SS	11	156	o	
					155		
154.59 24.63	SILTY SAND (TILL), some clay, trace gravel Compact Grey	18	SS	10	154		
153.62 25.60	CLAYEY SILT (TILL), trace sand, trace gravel Stiff to firm Grey	19	SS	11	153		
					152		
		20	SS	6	151		
		21	SS	NR ✳	150		
149.35							

Continued Next Page

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

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RECORD OF BOREHOLE No 102

METRIC

PROJECT 06-1130-180-0-3
G.W.P. 3031-06-00
LOCATION N 4685110.2 ;E 274370.2
ORIGINATED BY MA/BG
DIST HWY
BOREHOLE TYPE POWER AUGER / HOLLOW STEM & MUD ROTARY
COMPILED BY BRS
DATUM GEODETIC
DATE October 23, 2007 - October 26, 2007
CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
29.87	CLAYEY SILT (TILL), some sand, trace gravel Hard Grey	22	SS	47	Grout	149						
147.83 31.39	SILT, trace sand, trace clay, trace gravel Dense Grey	23	SS	49		148, 147			4	2	87	7
146.30 32.92	SILTY SAND (TILL), some gravel, trace clay, hydrogen sulphide odour Very dense Grey	24	SS	74	Bentonite	146			14	48	30	8
145.23 33.99	LIMESTONE bedrock Grey (Dundee Formation)	25	WS	-	Sand Fill	145						
		26	WS	-	Bentonite							
144.14 35.08	END OF BOREHOLE	27	SS	100/ 25mm								

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
REMARKS & GRAIN SIZE DISTRIBUTION (%)

Borehole dry during drilling.

Groundwater measured at elev. 178.20m in deep piezometer on November 20, 2007.

Groundwater measured at elev. 177.82m in shallow piezometer on November 20, 2007.

✳ Blow counts were not recorded (NR)

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

LDN_MTO_01 06-1130-180-0-3.GPJ LDN_MTO.GDT 2/11/08

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 103

METRIC

PROJECT 06-1130-180-0-3
G.W.P. 3031-06-00
LOCATION N 4685326.7 ;E 273035.6
ORIGINATED BY MA
DIST ____ HWY ____
BOREHOLE TYPE POWER AUGER / HOLLOW STEM & MUD ROTARY
COMPILED BY BRS
DATUM GEODETIC
DATE November 1, 2007 - November 5, 2007
CHECKED BY ____

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
181.63	GROUND SURFACE
0.00	TOPSOIL, silty
0.15	Brown
0.33	(FILL), sand and gravel, crushed, some silt Compact Brown
180.26 / 1.37	(FILL), clayey silt, trace sand, trace gravel, trace topsoil Stiff Brown and grey
	CLAYEY SILT (TILL), some sand, trace gravel Stiff to hard Brown becoming grey below about elev. 177.21m
172.24 / 9.39	SANDY SILT (TILL), trace gravel Grey
171.88 / 9.75	CLAYEY SILT (TILL), trace sand, trace gravel Stiff Grey
171.57 / 10.06	SANDY SILT (TILL), some clay, trace gravel Very dense to compact Grey
169.10 / 12.53	CLAYEY SILT (TILL), some sand, trace gravel Firm to stiff Grey

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	Water content / Remarks	Unit weight γ (kN/m³)	GR	SA	SI	CL
1	SS	11						
2	SS	10						
3	SS	22						
4	SS	37						
5	SS	31			2	24	41	33
6	SS	21						
7	SS	14						
8	SS	18						
9	SS	10						
10	SS	10						
11	TO	PH	Consolidation	21.5	3	33	43	21
12	SS	51						
13	SS	54			3	41	39	17
14	SS	19						
15	SS	8						

Ground water conditions: Bentonite (top); Grout (below about elev. 174)

Elevation scale: 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167

Shear strength (field vane) near elev. 173: 136 (sensitivity 1.3); >144

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

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

LDN_MTO_01 06-1130-180-0-3.GPJ LDN_MTO.GDT 2/11/08

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 103

METRIC

PROJECT	06-1130-180-0-3				
G.W.P.	3031-06-00	LOCATION	N 4685326.7 ;E 273035.6	ORIGINATED BY	MA
DIST	HWY	BOREHOLE TYPE	POWER AUGER / HOLLOW STEM & MUD ROTARY	COMPILED BY	BRS
DATUM	GEODETIC	DATE	November 1, 2007 - November 5, 2007	CHECKED BY	

SOIL PROFILE | SAMPLES | 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 (%) 10 20 30 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
		16	TO	PH	166				
		17	TO	PH	165 / 164				
		18	TO	PH	163	3	20	42	35
		19	TO	PH	162				
		20	SS	5	161				
		21	SS	7	160				
		22	SS	7	159 / 158	2	24	37	37
		23	SS	8	157 / 156				
155.36 26.27	SILTY SAND (TILL), trace clay Compact Grey	24	SS	12	155				
154.96 26.67	CLAYEY SILT (TILL), some sand, trace gravel Firm to hard Grey	25	SS	8	154 / 153				
		26	SS	8	152				

Grout

Continued Next Page

+³, ×³: Numbers refer to Sensitivity   O³% STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 103

METRIC

PROJECT 06-1130-180-0-3
G.W.P. 3031-06-00
LOCATION N 4685326.7 ;E 273035.6
ORIGINATED BY MA
DIST ____ HWY ____
BOREHOLE TYPE POWER AUGER / HOLLOW STEM & MUD ROTARY
COMPILED BY BRS
DATUM GEODETIC
DATE November 1, 2007 - November 5, 2007
CHECKED BY ____

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
145.66 / 35.97	SILT, some sand Dense Grey
144.14 / 37.49	CLAYEY SILT (TILL), some sand, some gravel Hard Grey
142.62 / 39.01	SILTY SAND (TILL), trace clay, some gravel Very dense Grey
140.63 / 41.00	LIMESTONE BEDROCK (inferred) (Dundee Formation)
139.57 / 42.06	END OF BOREHOLE

Groundwater encountered at about elev. 145.7m depth during drilling no November 2, 2007.

Groundwater measured in deep piezometer at elev. 179.06m on November 20, 2007.

SAMPLES

NUMBER	TYPE	"N" VALUES	Approx. elevation (m)	Water content / limits	GR	SA	SI	CL
27	SS	9	151	w_P – w – w_L plotted	1	22	40	37
28	SS	5	149.5					
29	SS	7	148	w plotted				
30	SS	40	146.5					
31	SS	49	145					
32	SS	54	143.5	w – w_P – w_L plotted	11	30	42	17
33	SS	100/ 150mm	142					
~~34~~	~~SS~~	100/ 25mm	140.5					
~~35~~	~~SS~~	100/ 0mm	139.5					

GROUND WATER CONDITIONS

Grout (to approx. elev. 144.2); Bentonite (approx. elev. 144.2 – 143.9); Sand Fill (below); groundwater level symbol at approx. elev. 145.7.

Column 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/m^3
REMARKS & GRAIN SIZE DISTRIBUTION (%)	GR SA SI CL

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 103A

METRIC

PROJECT 06-1130-180-0-3

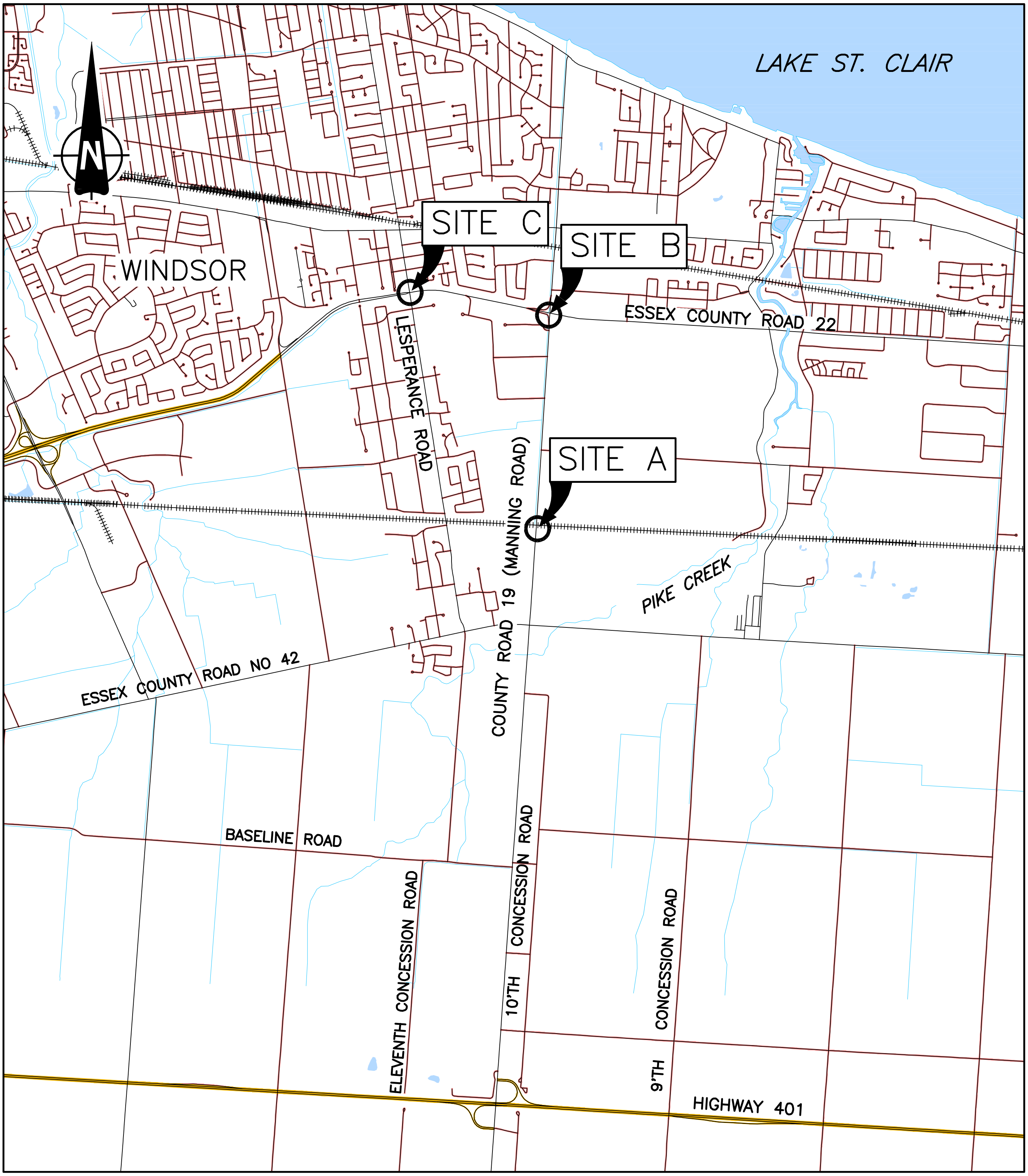
G.W.P. 3031-06-00 LOCATION N 4685326.7 ;E 273035.6 ORIGINATED BY MA

DIST HWY BOREHOLE TYPE POWER AUGER / HOLLOW STEM & MUD ROTARY COMPILED BY LMK/BRS

DATUM GEODETIC DATE November 5, 2007 CHECKED BY

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	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 (○ 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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.63	GROUND SURFACE										
0.00	REFER TO BOREHOLE 103 FOR SOIL STRATIGRAPHY					Cuttings	181				
						Bentonite	180				
							179				
							178				
						Cuttings	177				
						▼	176				
							175				
						Bentonite	174				
							173				
						Cuttings	172				
						Sand Fill	171				
169.44							170				
12.19	END OF BOREHOLE Groundwater measured in shallow piezometer at elev. 176.27m on November 20, 2007.										

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



SCALE IN METRES

0 1000 2000

1:50,000

PROJECT

GRADE SEPARATIONS
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
GWP 3031-06-00

TITLE

KEY PLAN

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-180-0-3		FILE No.	0611301800-3-F01001
			SCALE AS SHOWN	REV. 0
CADD	DCH	Feb. 15/08	FIGURE 1	
CHECK				

DRAWING FILE: 0611301800-3-F01001.DWG Plot Date: Feb 15, 2008 - 11:05am

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

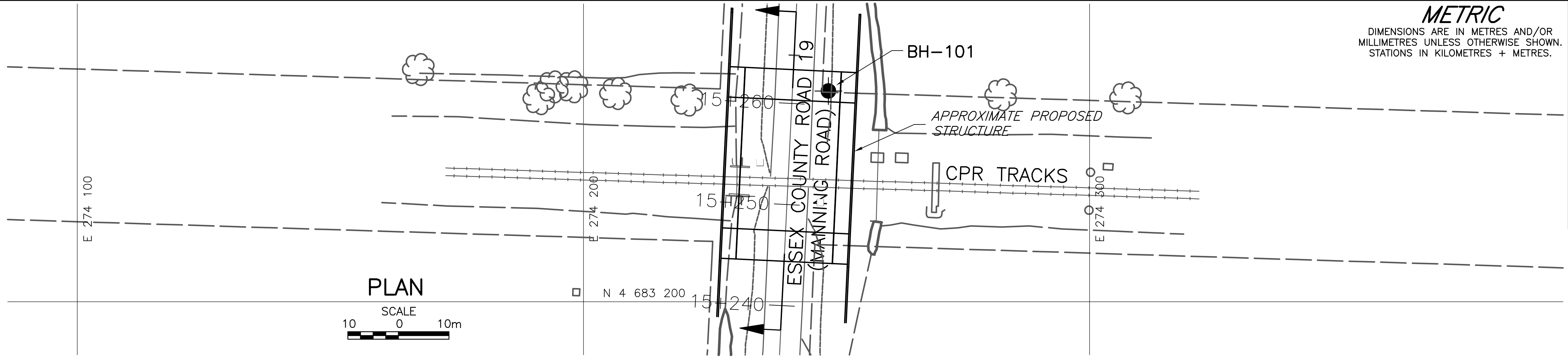
CONT No.
WP No. 3031–06–00

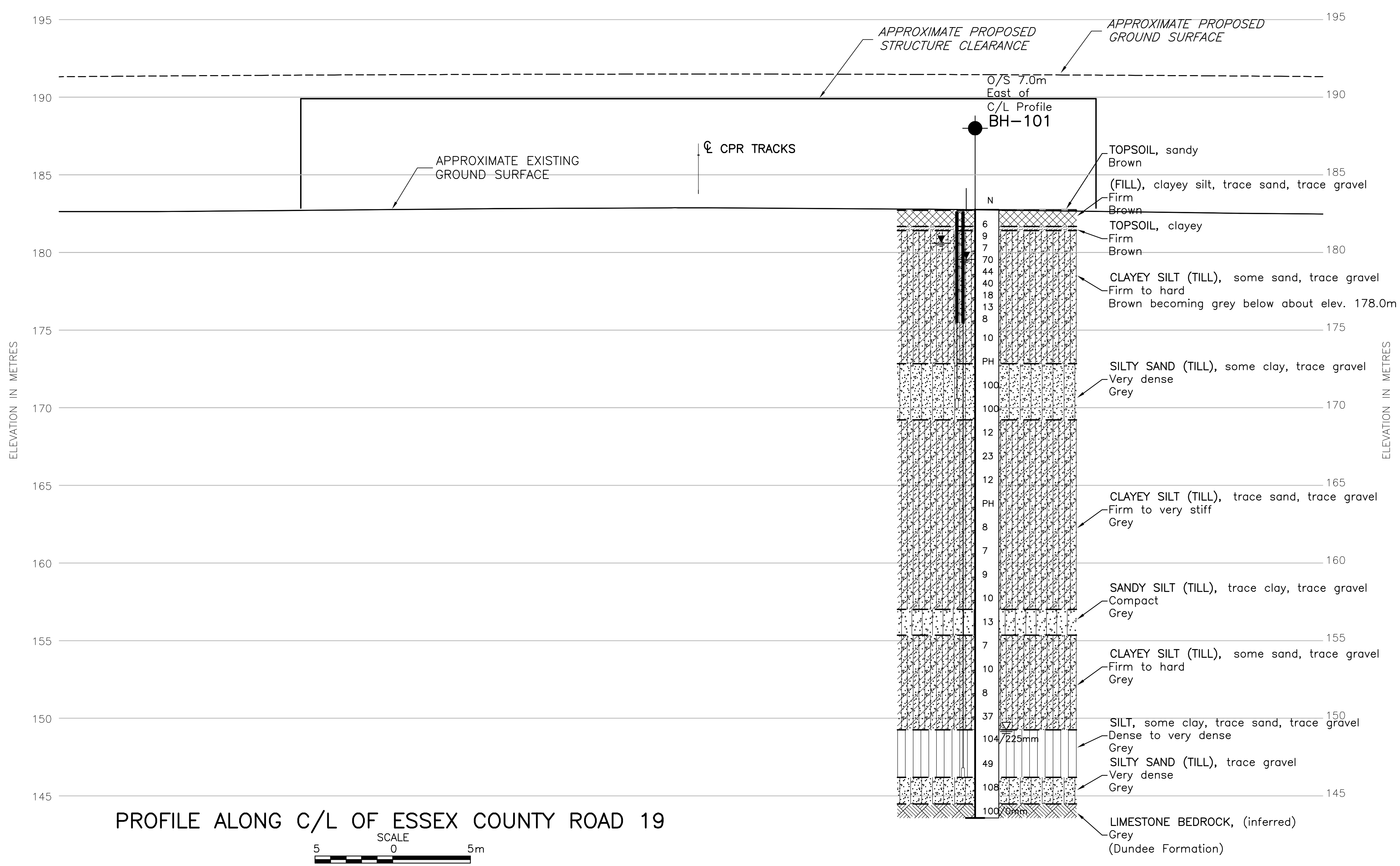
SHEET

MANNING ROAD WIDENING
ESSEX COUNTY ROAD 19/
CPR GRADE SEPARATION
BOREHOLE LOCATION & SOIL STRATA

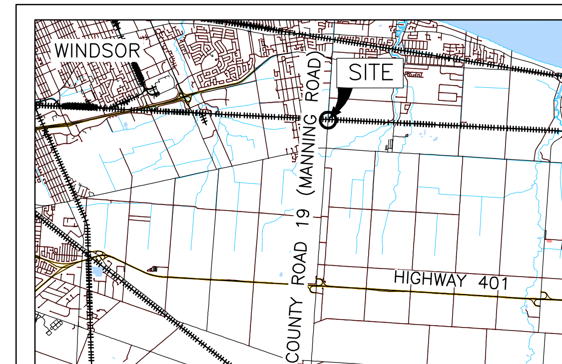
Golder Associates

Golder Associates Ltd.
LONDON, ONTARIO, CANADA





PROFILE ALONG C/L OF ESSEX COUNTY ROAD 19
SCALE
5 0 5m



KEY PLAN
SCALE
2000 0 2000m

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometers, measured on February 14, 2008
- WL encountered during drilling

No.	ELEVATION	CO–ORDINATES (MTM Zone 11)	
		NORTHING	EASTING
101	182.78	4 683 241.7	274 248.4

NOTES

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

REFERENCE

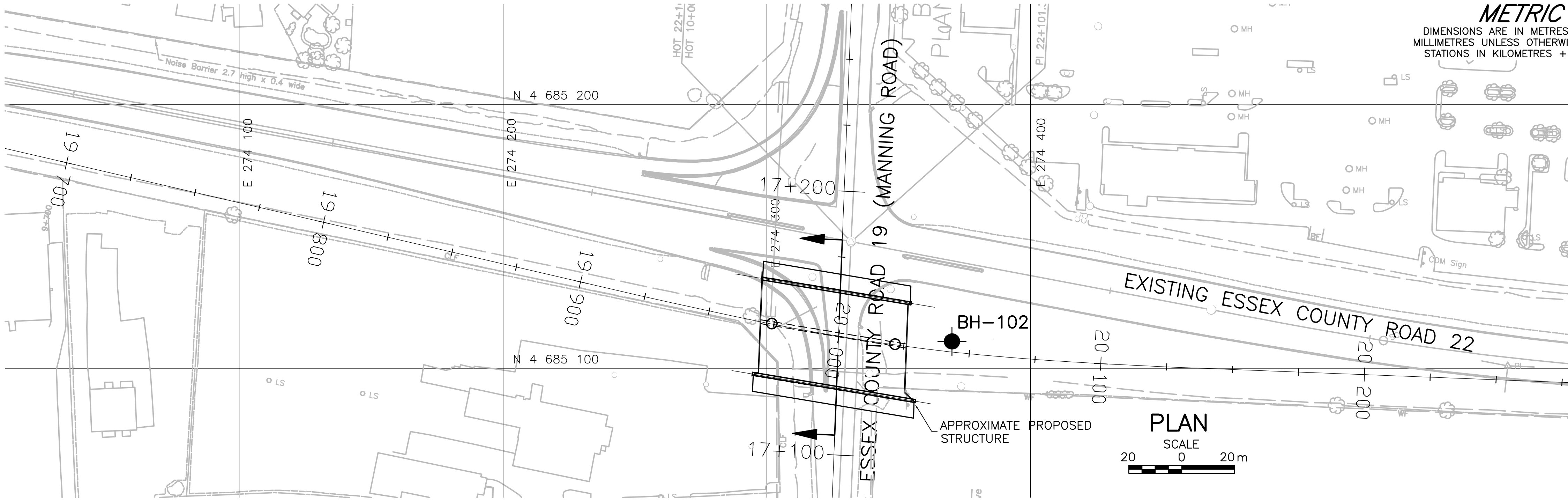
Preliminary base plans provided in digital format by Dillon.

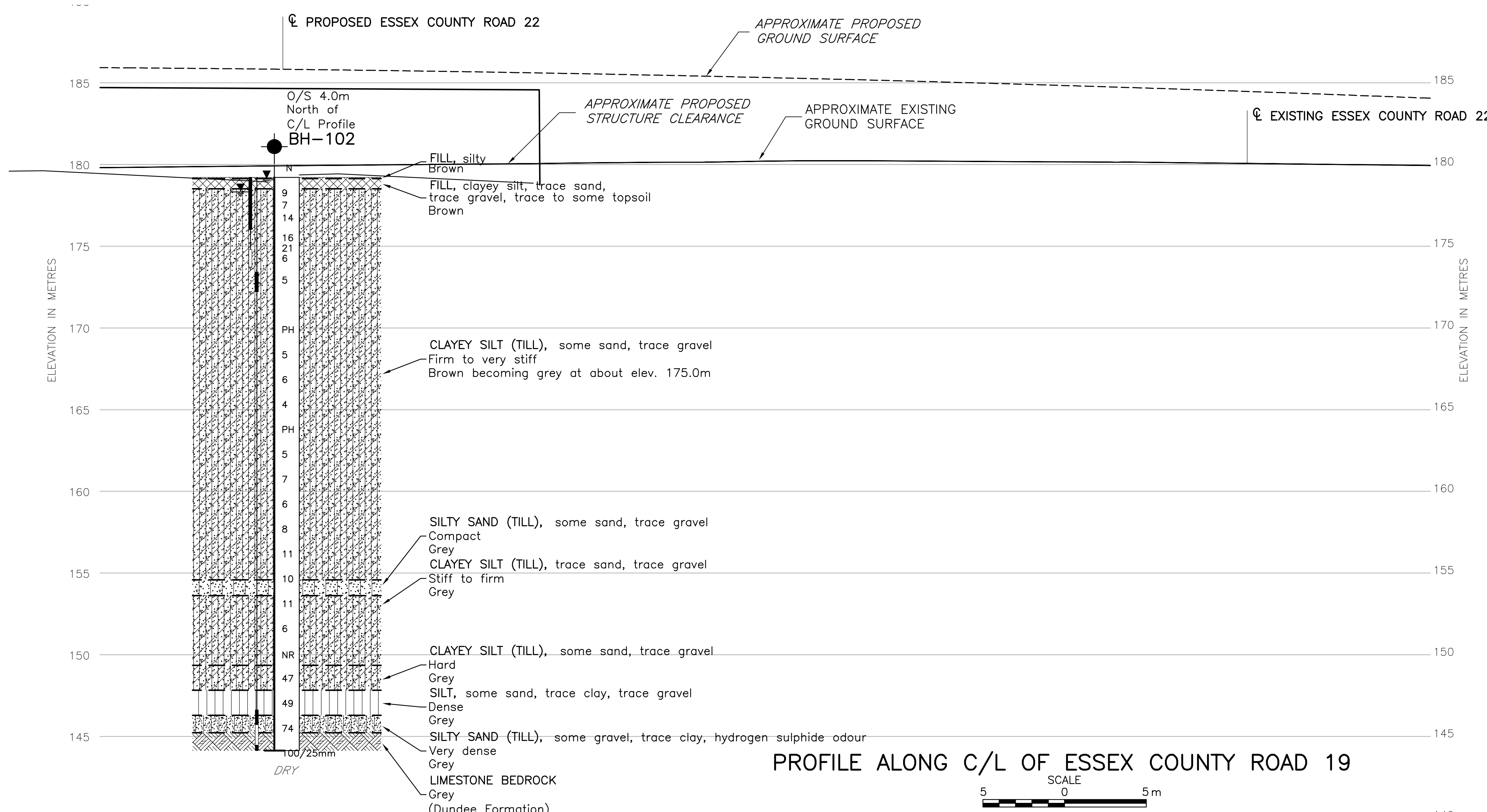
NO.	DATE	BY	REVISION

Geocres No.	40J7–23				
HWY.				PROJECT NO. 06–1130–180–0–3	DIST.
SUBM'D.	SMG	CHKD.	DUP	DATE: Feb. 08/08	SITE: B–19–04
DRAWN:	DCH	CHKD.	DUP	APPD.	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

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





PROFILE ALONG C/L OF ESSEX COUNTY ROAD 19

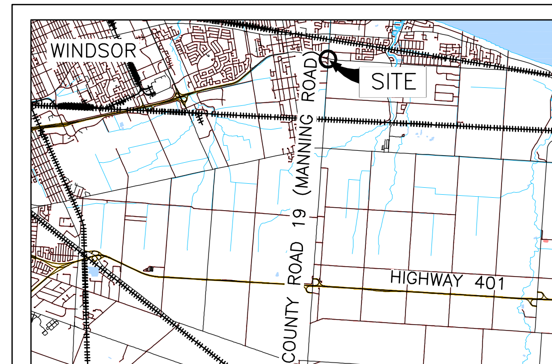
SCALE
5 0 5m

CONT No.
WP No. 3031-06-00

SHEET

MANNING ROAD WIDENING
ESSEX Co. RD 19/ESSEX Co. RD 22
GRADE SEPARATION
BOREHOLE LOCATION & SOIL STRATA

Golder Associates
Golder Associates Ltd.
LONDON, ONTARIO, CANADA



KEY PLAN
SCALE
2000 0 2000m

LEGEND

- ● Borehole – Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured on February 14, 2008
- DRY Borehole dry during drilling
- NR Blow counts were not recorded

No.	ELEVATION	CO-ORDINATES (MTM Zone 11)	
		NORTHING	EASTING
102	179.22	4 685 110.2	274 370.2

NOTES

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

REFERENCE

Preliminary base plans provided in digital format by Dillon.

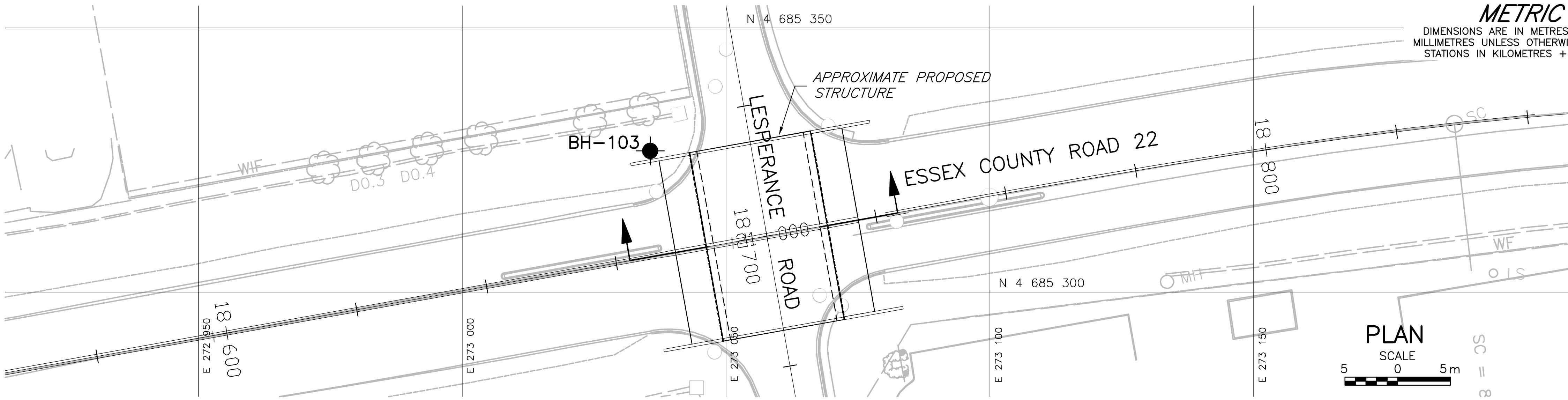
NO.	DATE	BY	REVISION

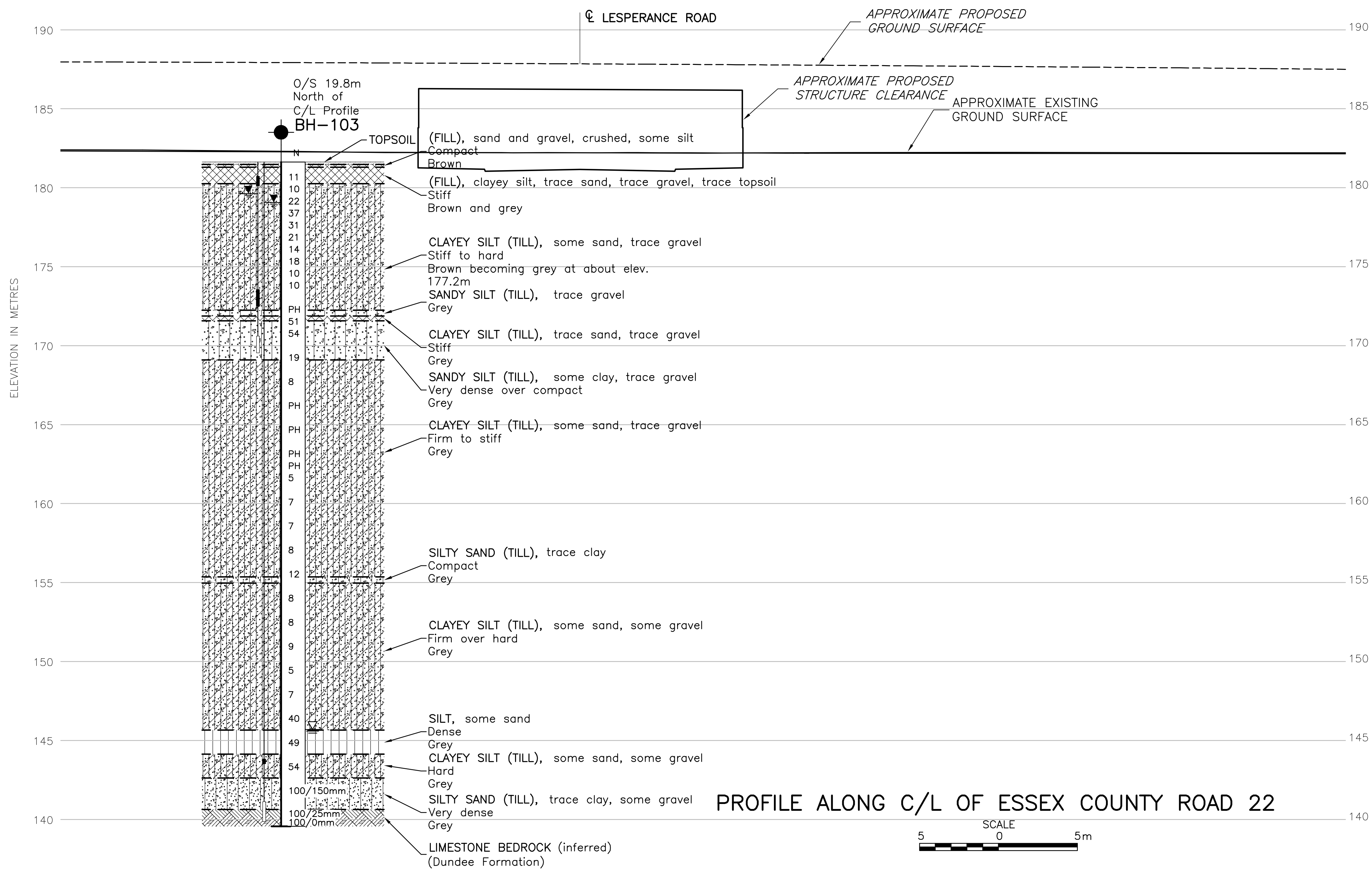
Geocres No. 40J7-23

HWY. 19		PROJECT NO. 06-1130-180-0-3		DIST.	
SUBM'D. SMG	CHKD. DUP	DATE: Feb. 08/08		SITE: B-22-03	
DRAWN: DCH	CHKD. DUP	APPD.		DWG. 2	

PLOT DATE: February 15, 2008

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





PROFILE ALONG C/L OF ESSEX COUNTY ROAD 22

SCALE
5 0 5m

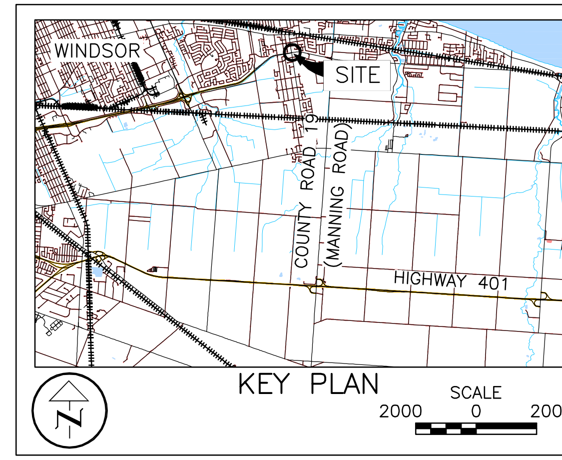
CONT No.
WP No. 3031-06-00

SHEET

HIGHWAY 2 WIDENING
ESSEX COUNTY ROAD 22 (HWY 2)/
LESPERANCE RD. GRADE SEPARATION
BOREHOLE LOCATION & SOIL STRATA

Golder Associates

Golder Associates Ltd.
LONDON, ONTARIO, CANADA



KEY PLAN
SCALE
2000 0 2000m

LEGEND

●	Borehole – Current Investigation
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▼	WL in piezometer, measured on February 14, 2008
▽	WL encountered during drilling

No.	ELEVATION	CO-ORDINATES (MTM Zone 11)	
		NORTHING	EASTING
103	181.63	4 685 326.7	273 035.6

NOTES

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

REFERENCE

Preliminary base plans provided in digital format by Dillon.

NO.	DATE	BY	REVISION

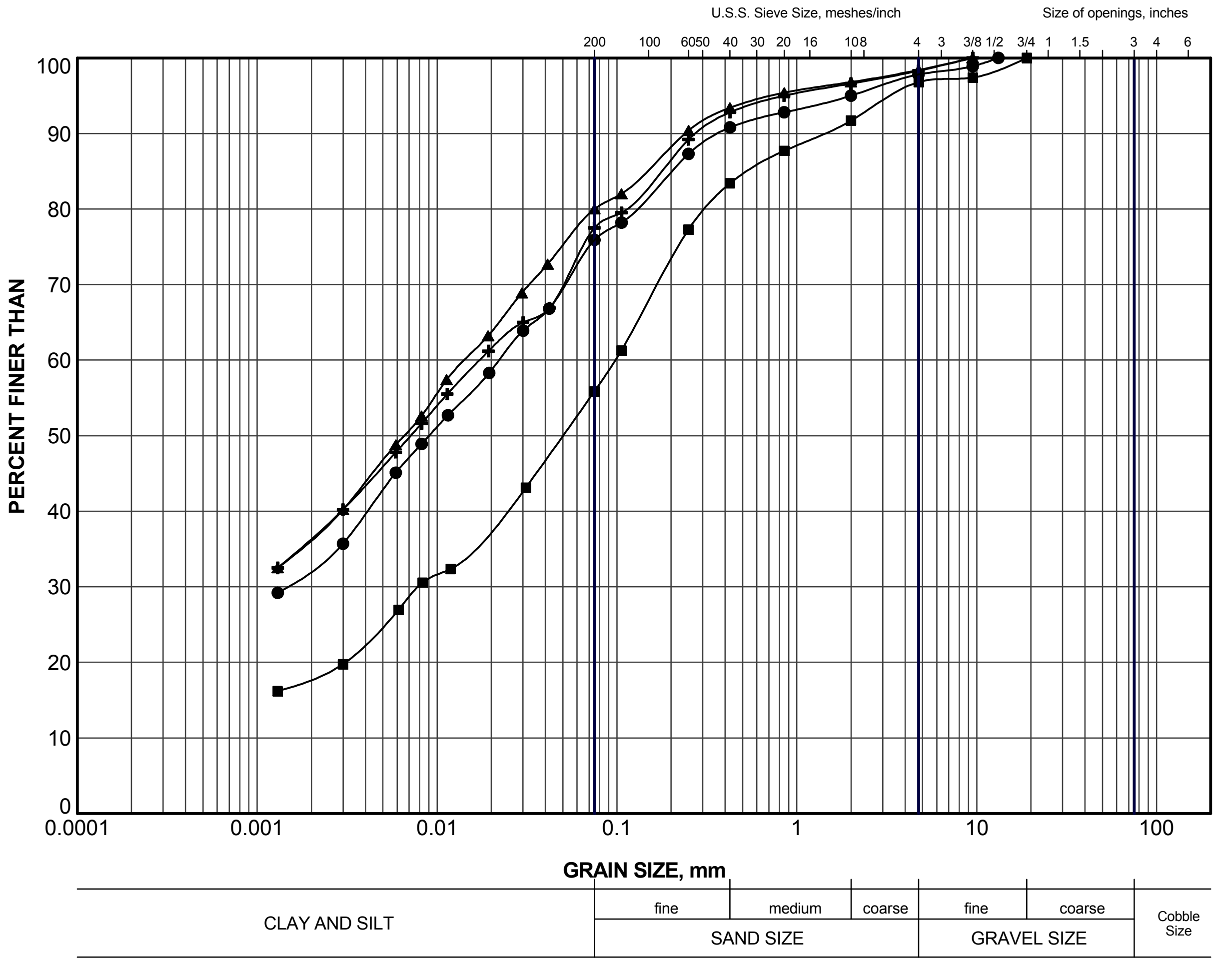
Geocres No. 40J7-23

HWY.		PROJECT NO. 06-1130-180-0-3	DIST.
SUBM'D. SMG	CHKD. DUP	DATE: Feb. 08/08	SITE: 22-02
DRAWN: DCH	CHKD. DUP	APPD.	DWG. 3

MINISTRY OF TRANSPORTATION, ONTARIO

APPENDIX A

ESSEX COUNTY ROAD 19 AND CPR (SITE A) LABORATORY TEST DATA



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	101	5	178.7
■	101	11	173.0
▲	101	18	162.3
✚	101	23	154.7

PROJECT
CANADIAN PACIFIC RAILWAY OVERHEAD
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

TITLE

GRAIN SIZE DISTRIBUTION

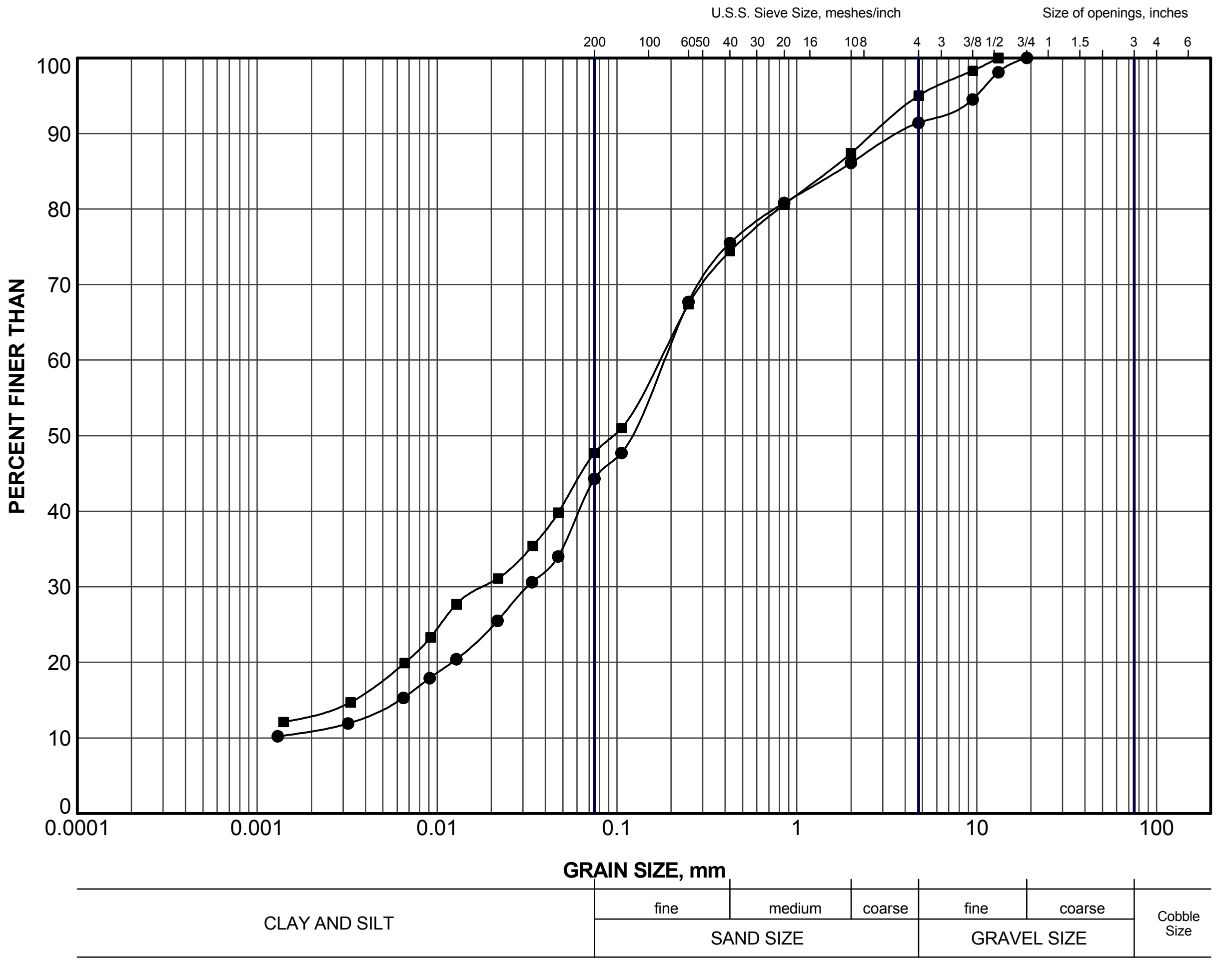
CLAYEY SILT (TILL)

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-180-0-3	FILE No.	0611301800-3-D010A1
		SCALE N/A	REV.
DRAWN	DCH	Feb. 08/08	
CHECK			

FIGURE A-1

LDN_MTO_NEW GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	101	12	171.4
■	101	29	145.5

PROJECT
CANADIAN PACIFIC RAILWAY OVERHEAD
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

TITLE

GRAIN SIZE DISTRIBUTION

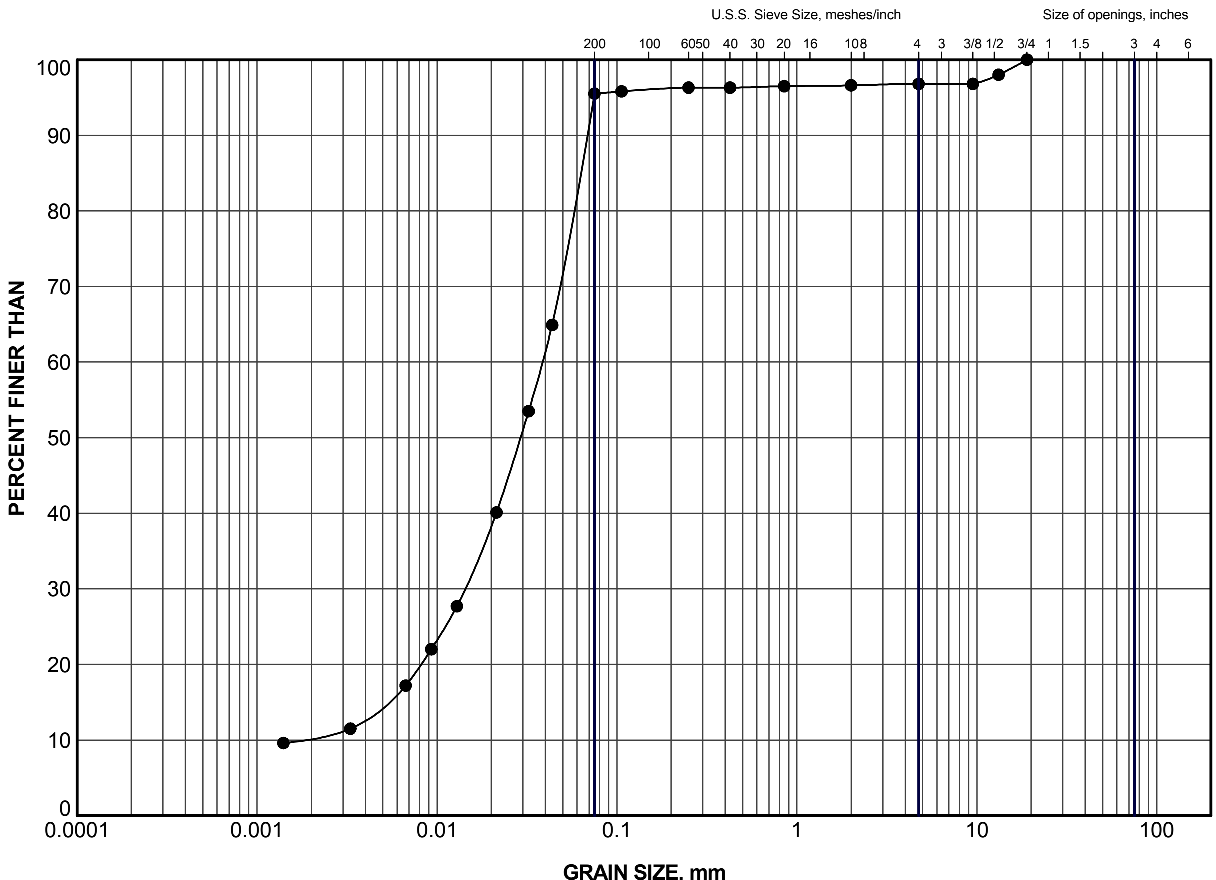
SILTY SAND (TILL)

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-180-0-3	FILE No.	0611301800-3-D010A2
		SCALE	N/A
DRAWN	BRS	Feb. 08/08	REV.
CHECK			

FIGURE A-2

LDN_MTO_NEW GLDR_LDN.GDT



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	101	27	148.7

PROJECT

CANADIAN PACIFIC RAILWAY OVERHEAD
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

TITLE

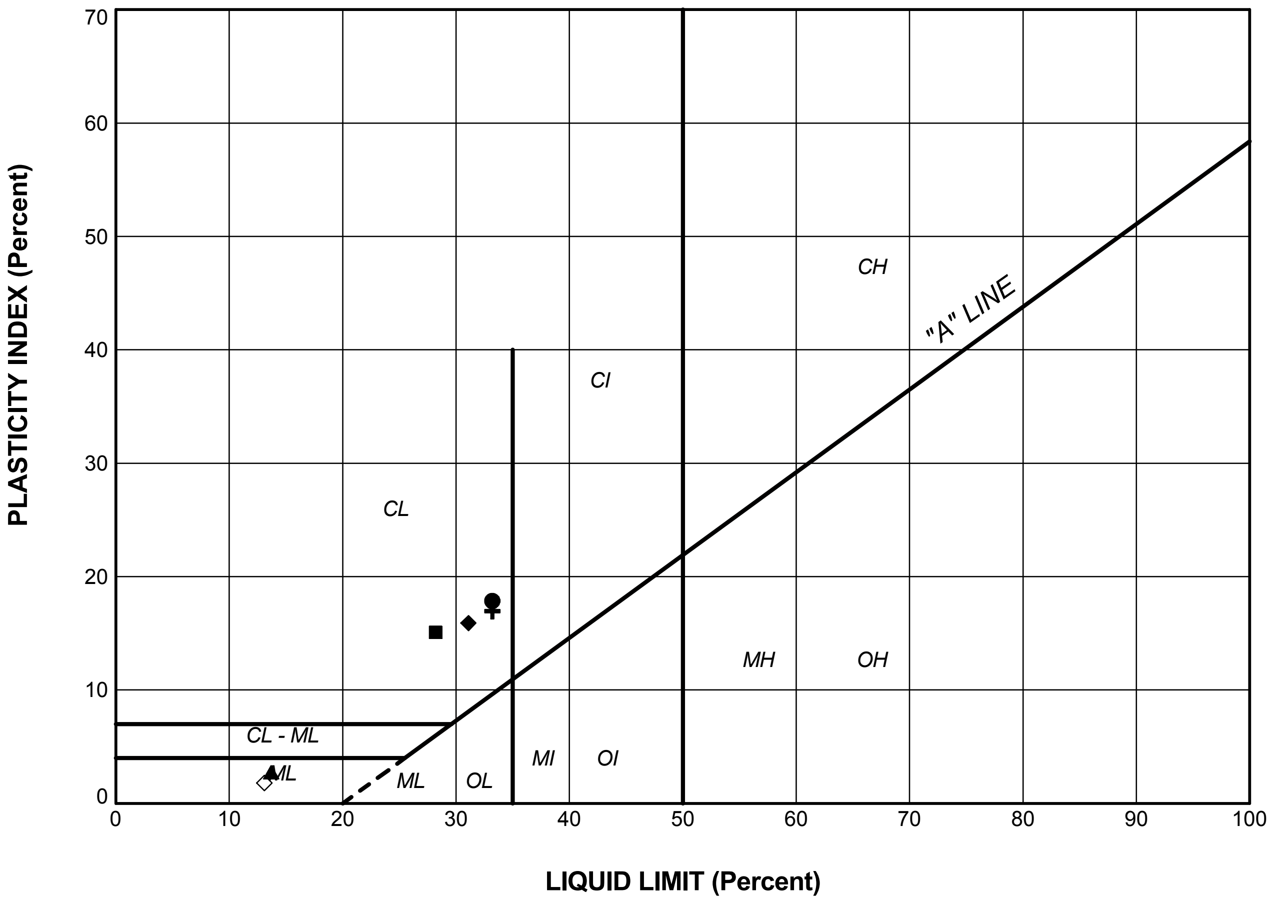
GRAIN SIZE DISTRIBUTION

SILT

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-180-0-3		FILE No. 0611301800-3-D010A3	
			SCALE N/A	REV.
DRAWN	BRS	Feb. 08/08	FIGURE A-3	
CHECK				

LDN_MTO_NEW GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
CLAYEY SILT (TILL)					
●	101	5	33.2	15.4	17.9
■	101	11	28.2	13.1	15.1
✚	101	18	33.2	16.3	17.0
◆	101	23	31.1	15.2	15.9
SILTY SAND (TILL)					
▲	101	12	13.7	11.0	2.8
◇	101	29	13.1	11.3	1.8

PROJECT
CANADIAN PACIFIC RAILWAY OVERHEAD
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

TITLE

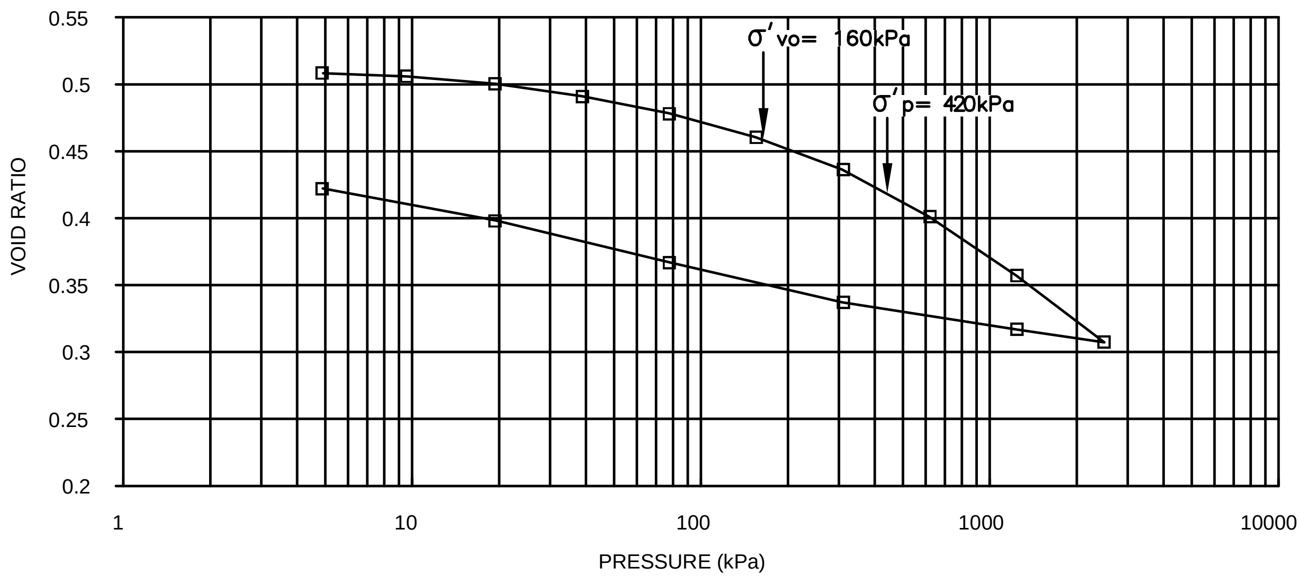
PLASTICITY CHART

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-180-0-3		FILE No. 0611301800-3-D010A4	
			SCALE N/A	REV.
DRAWN	BRS	Feb. 08/08	**FIGURE A-4**	
CHECK				

MTO_PI GLDR_LDN.GDT

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 101 SA 11



Drawing file: 0611301800-3-D010A5.dwg Feb 11, 2008 - 3:00pm

PROJECT

CANADIAN PACIFIC RAILWAY OVERHEAD
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

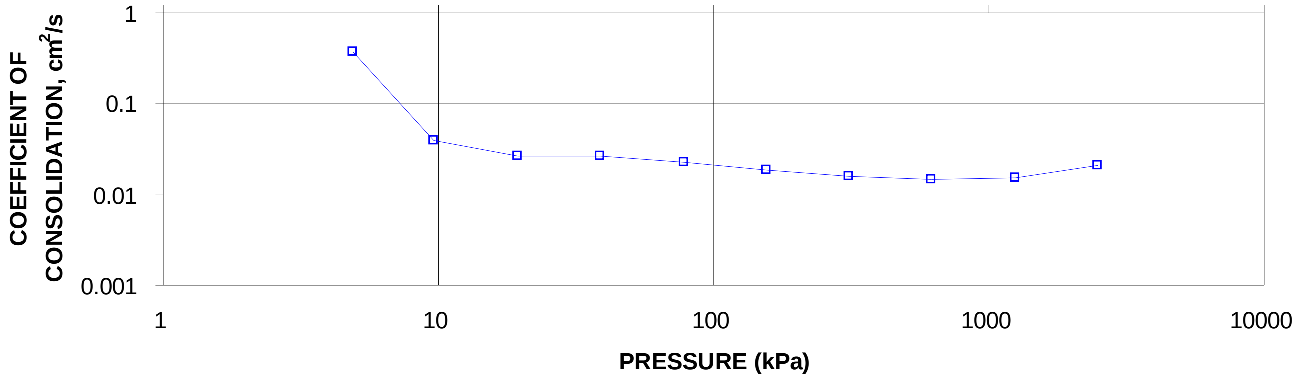
TITLE

CONSOLIDATION TEST

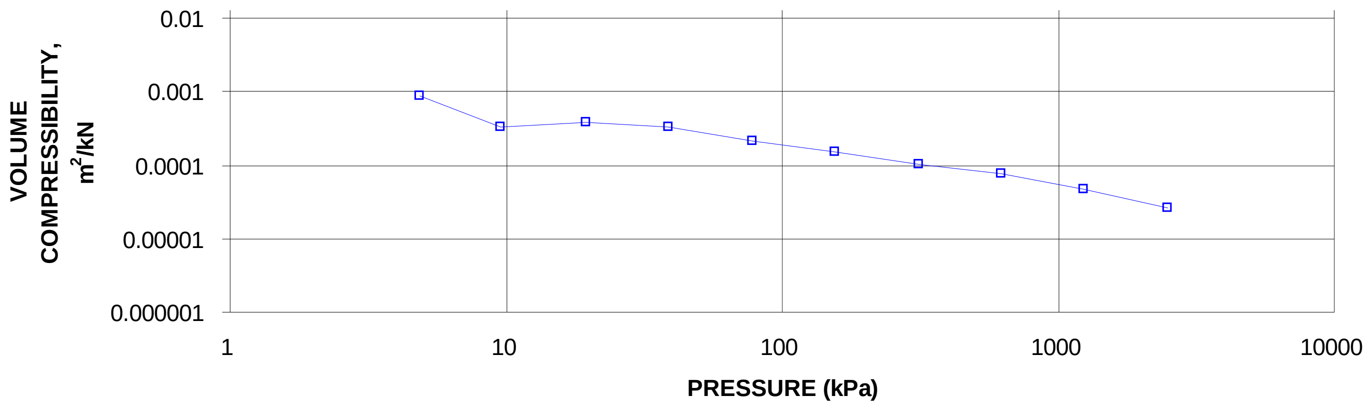
Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-180-0-3		FILE No.	0611301800-3-R010A5
			SCALE AS SHOWN	REV. 0
CADD	DCH	Feb. 08/08		
CHECK			FIGURE A-5	

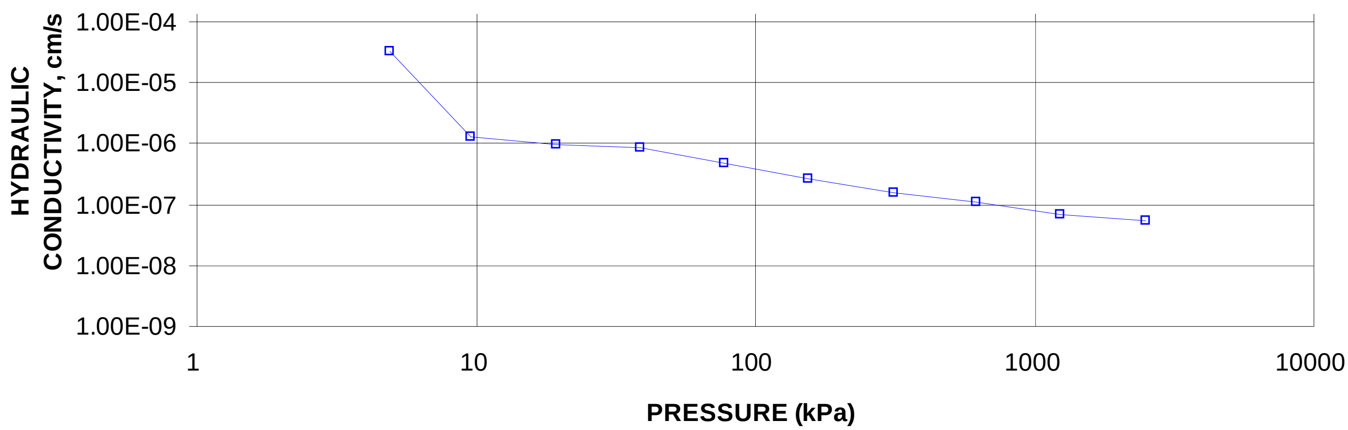
CONSOLIDATION TEST
CV cm²/s vs PRESSURE (kPa)
BH 101 SA 11



CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH 101 SA 11



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 101 SA 11



PROJECT
CANADIAN PACIFIC RAILWAY OVERHEAD
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

TITLE

OEDOMETER CONSOLIDATION SUMMARY

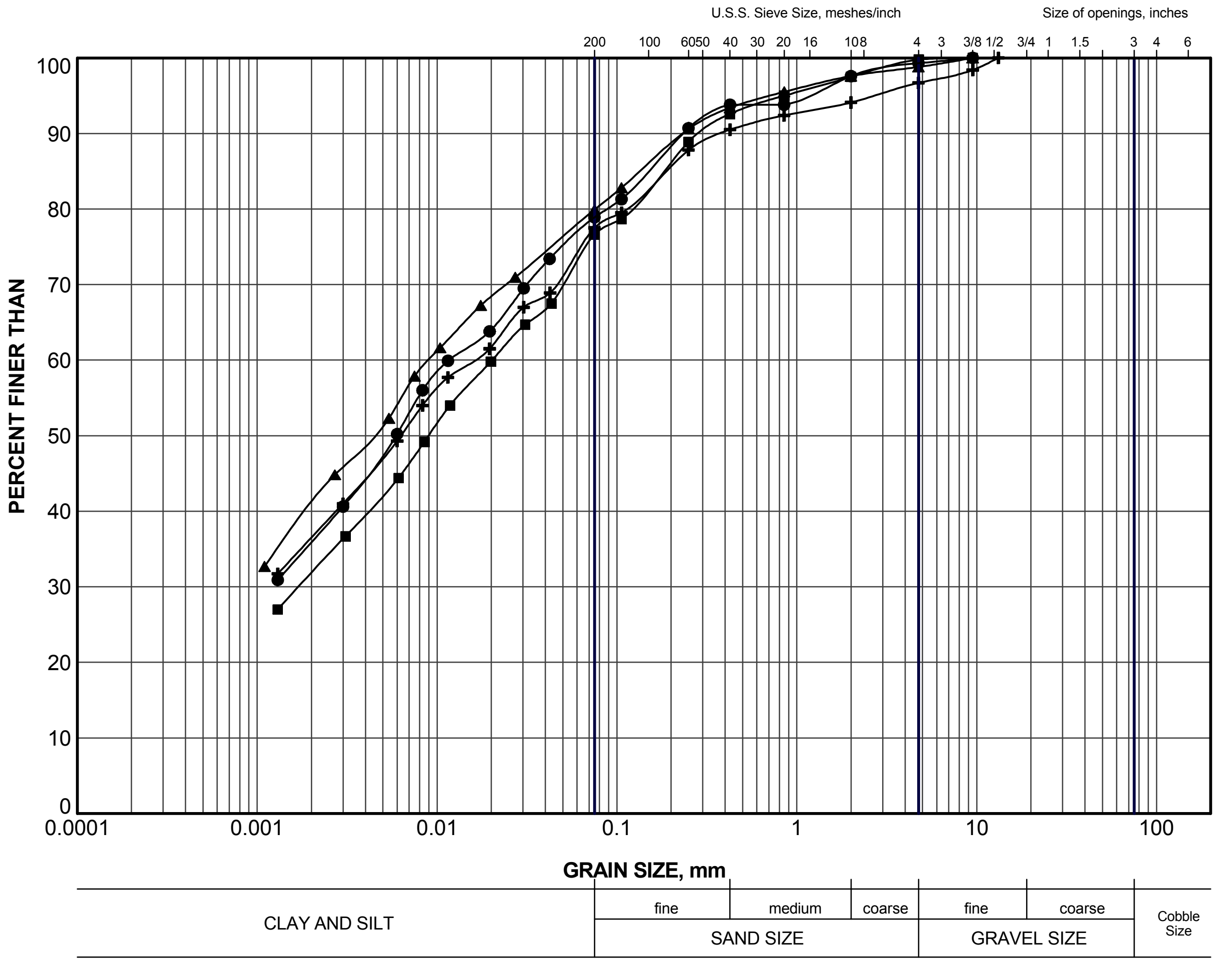
Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-180-0-3		FILE No.	0611301800-3-D010A6
			SCALE	AS SHOWN REV. 0
CADD	DCH	Feb. 08/08	**FIGURE A-6**	
CHECK				

DRAWING FILE: 0611301800-3-D010A6.DWG Plot Date: Feb 11, 2008 - 3:02pm

APPENDIX B

ESSEX COUNTY ROAD 19 AND
ESSEX COUNTY ROAD 22 (SITE B)
LABORATORY TEST DATA



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	102	3	176.7
■	102	6	174.4
▲	102	8	169.9
✚	102	15	159.2

PROJECT

COUNTY ROAD 22/MANNING ROAD UNDERPASS
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

TITLE

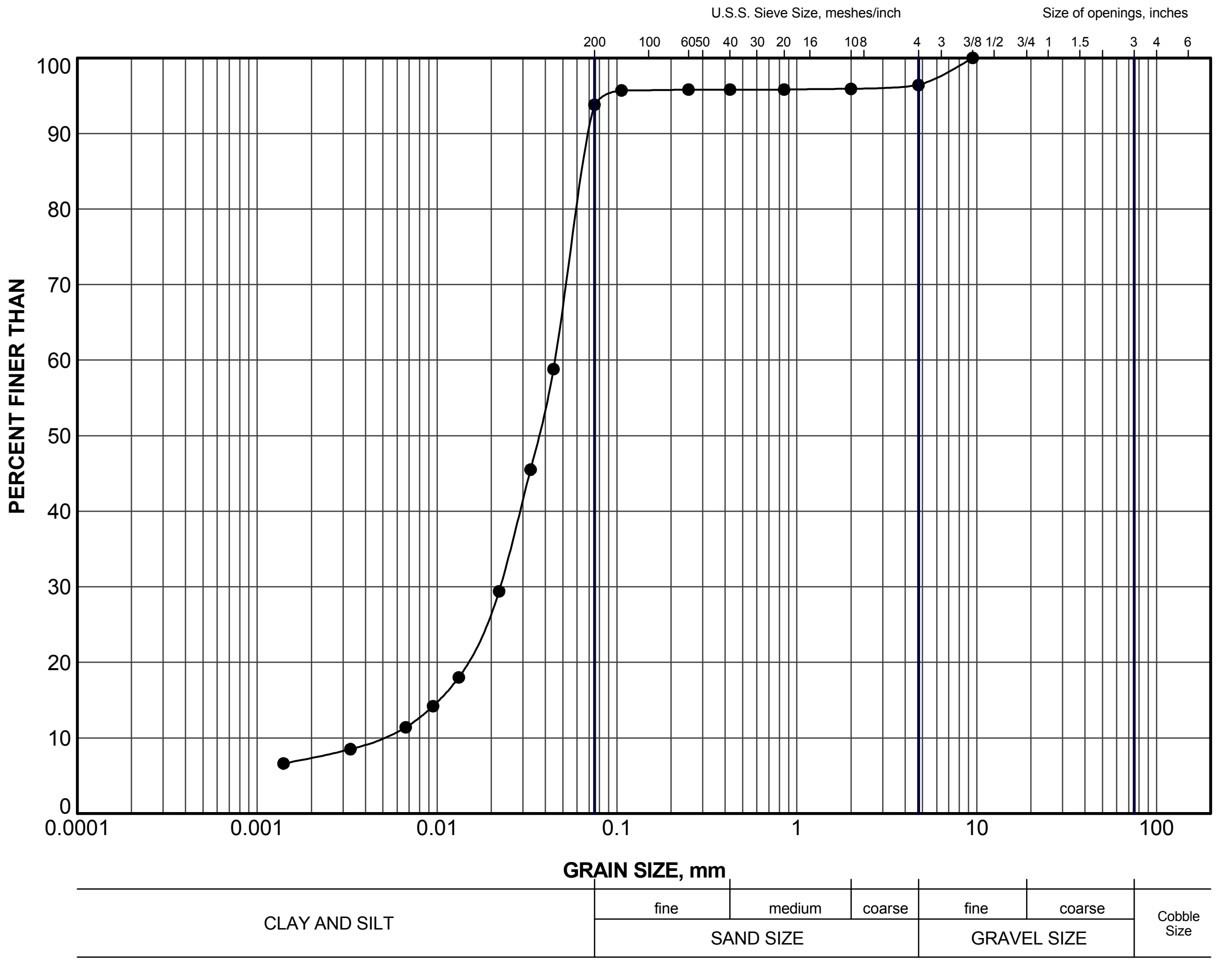
GRAIN SIZE DISTRIBUTION

CLAYEY SILT (TILL)

Golder Associates
LONDON, ONTARIO

PROJECT No. 06-1130-180-0-3			FILE No. 0611301800-3-F020B1
			SCALE N/A / REV.
DRAWN	DCH	Feb. 08/08	FIGURE B-1
CHECK			

LDN_MTO_NEW GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	102	23	147.0

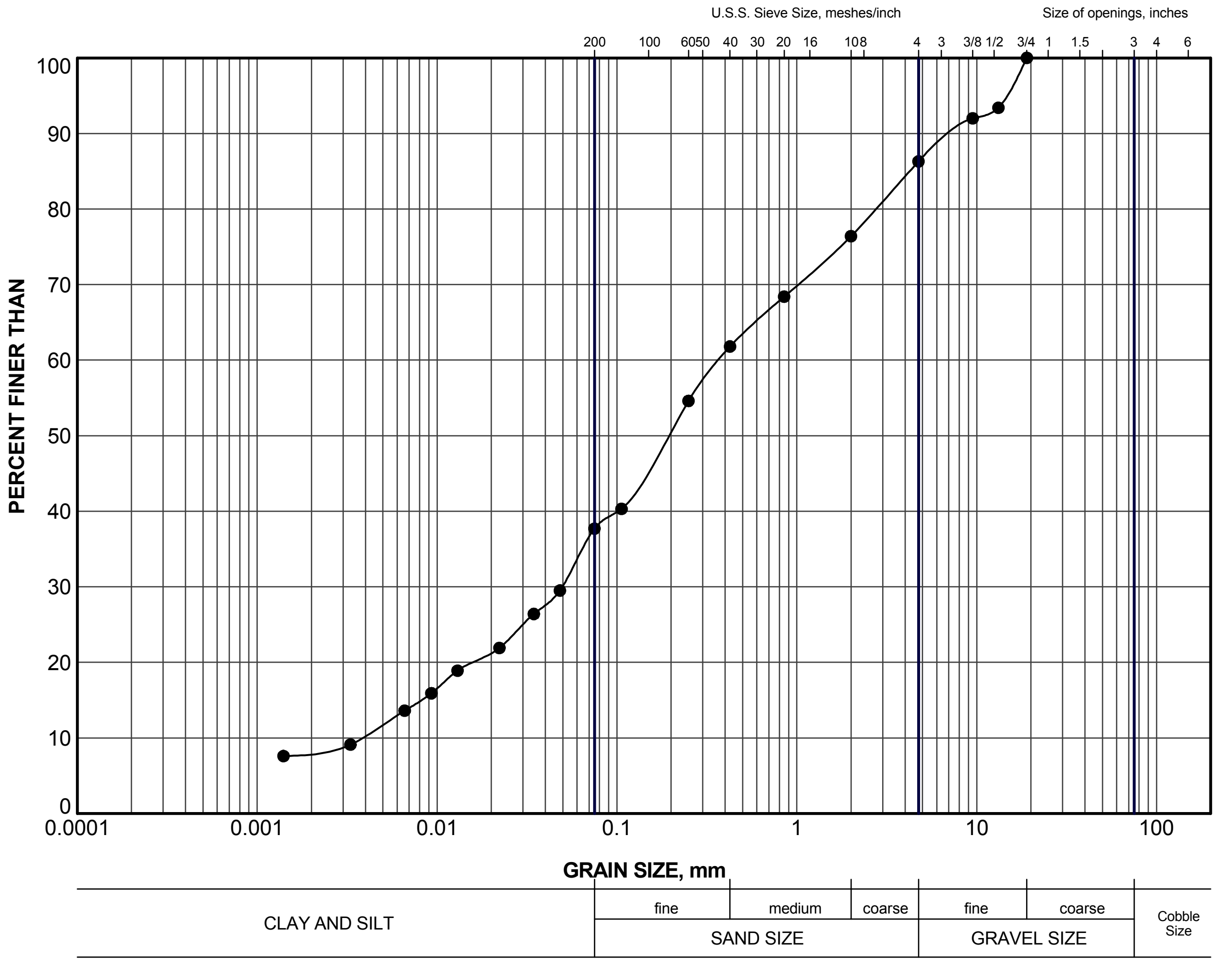
PROJECT
COUNTY ROAD 22/MANNING ROAD UNDERPASS
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

TITLE
GRAIN SIZE DISTRIBUTION
SILT

Golder Associates
LONDON, ONTARIO

PROJECT No. 06-1130-180-0-4			FILE No. 0611301800-3-F020B2
			SCALE N/A REV.
DRAWN	BRS	Feb. 08/08	**FIGURE B-2**
CHECK			

LDN_MTO_NEW GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	102	24	145.5

PROJECT

COUNTY ROAD 22/MANNING ROAD UNDERPASS
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

TITLE

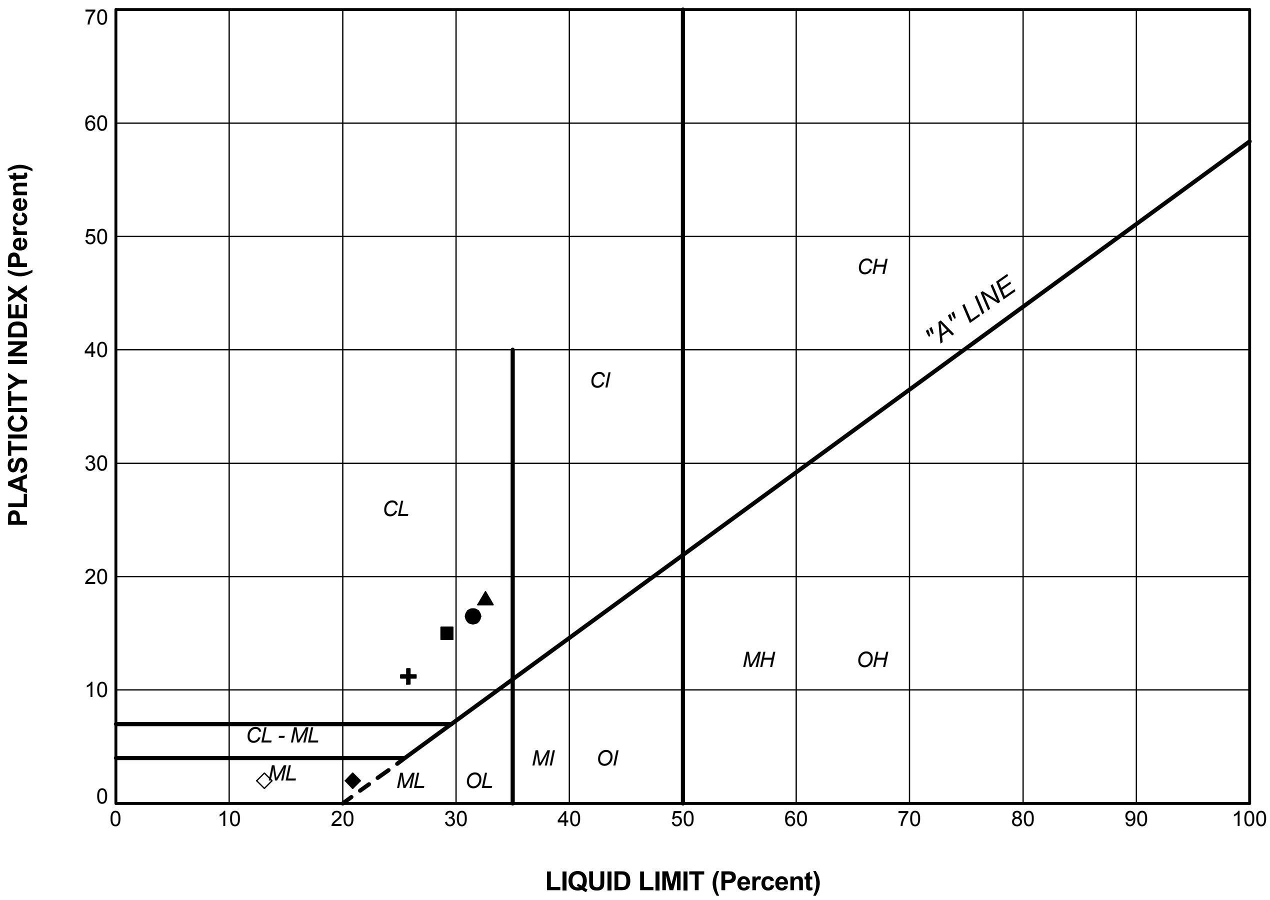
GRAIN SIZE DISTRIBUTION

SILTY SAND (TILL)

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-180-0-4	FILE No.	0611301800-3-F020B3
		SCALE N/A	REV.
DRAWN	BRS	Feb. 08/08	**FIGURE B-3**
CHECK			

LDN_MTO_NEW GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
CLAYEY SILT (TILL)					
●	102	3	31.5	15.0	16.5
■	102	6	29.2	14.2	15.0
▲	102	8	32.6	14.5	18.1
✚	102	15	25.8	14.6	11.2
SILTY SAND (TILL)					
◇	102	24	13.1	11.1	2.0
SILT					
◆	102	23	20.9	18.9	2.0

PROJECT
COUNTY ROAD 22/MANNING ROAD UNDERPASS
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

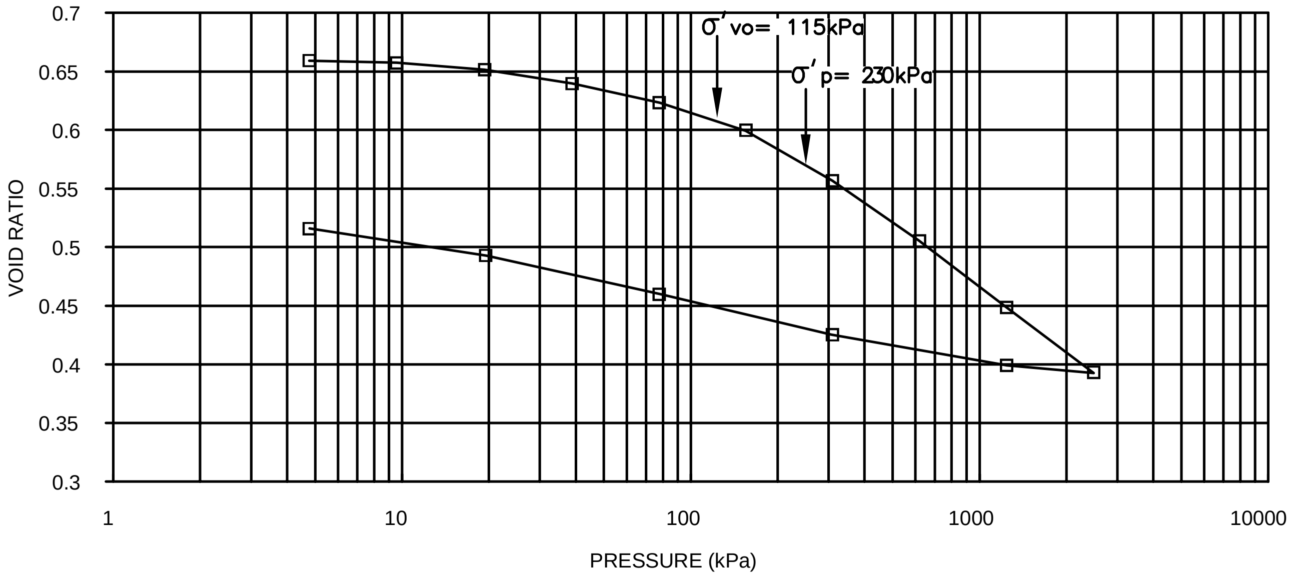
TITLE
PLASTICITY CHART

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-180-0-4	FILE No.	0611301800-3-F020B4
		SCALE N/A	REV.
DRAWN	BRS	Feb. 08/08	
CHECK			**FIGURE B-4**

MTO_PI GLDR_LDN.GDT

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 102 SA 8



PROJECT

COUNTY ROAD 22/MANNING ROAD UNDERPASS
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

TITLE

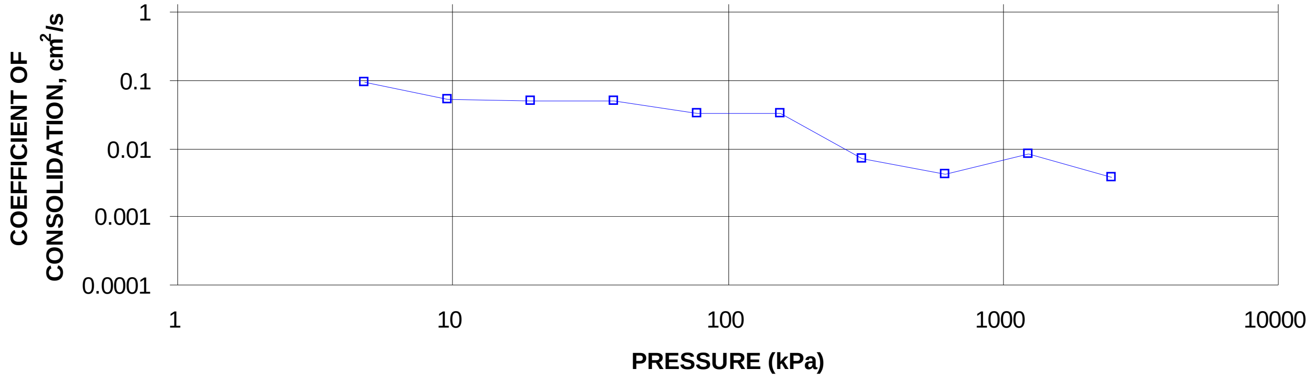
CONSOLIDATION TEST

Golder Associates
LONDON, ONTARIO

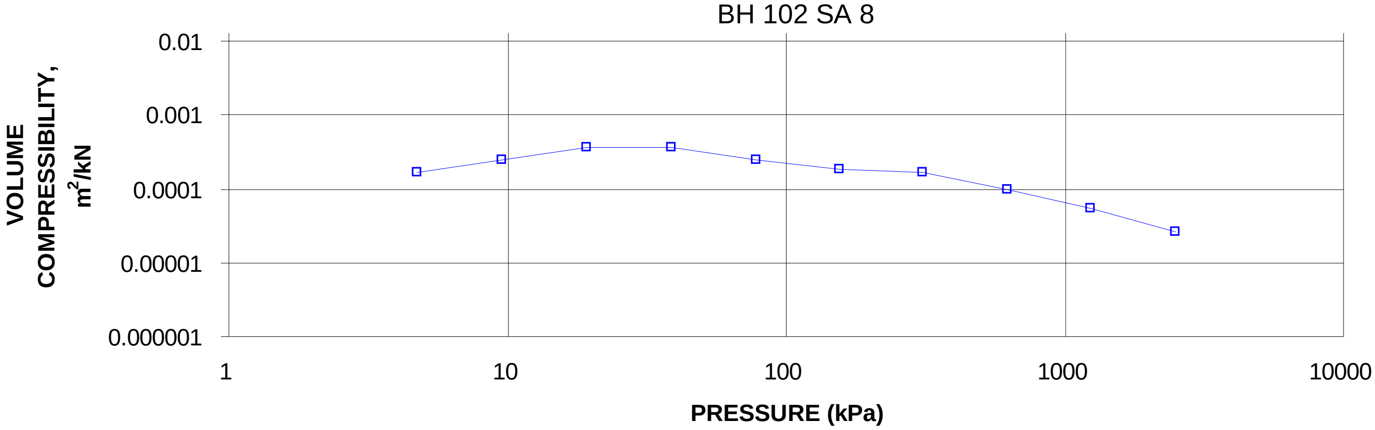
PROJECT No.	06-1130-180-0-3		FILE No.	0611301800-3-D020B5
			SCALE AS SHOWN	REV. 0
CADD	DCH	Feb. 08/08		
CHECK			**FIGURE B-5**	

Drawing file: 0611301800-3-D020B5.dwg Feb 11, 2008 - 3:03pm

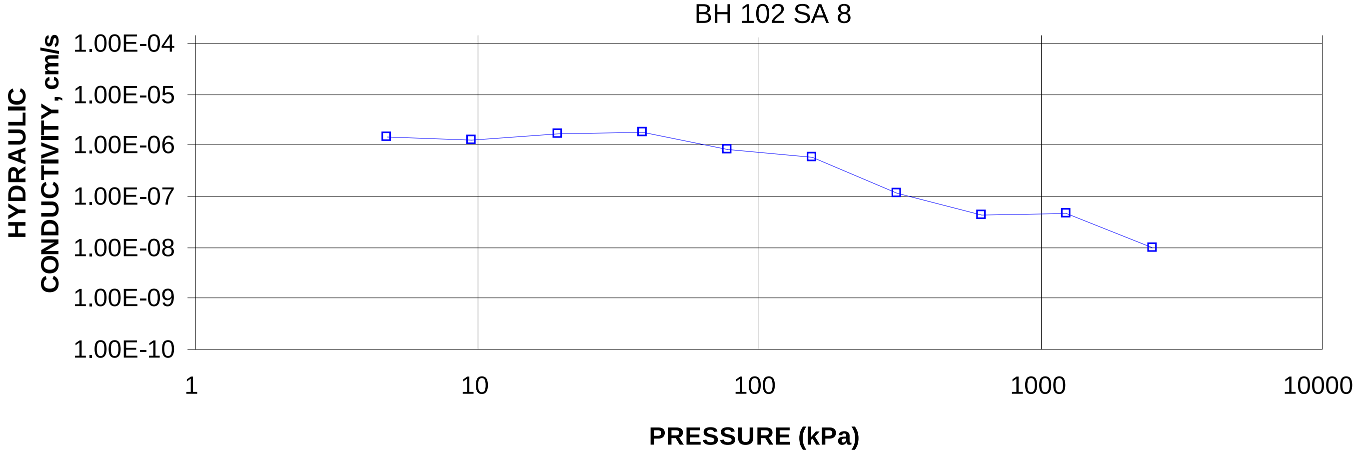
CONSOLIDATION TEST
CV cm²/s vs PRESSURE (kPa)
BH 102 SA 8



CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH 102 SA 8



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 102 SA 8



PROJECT
COUNTY ROAD 22/MANNING ROAD UNDERPASS
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

TITLE
OEDOMETER CONSOLIDATION SUMMARY

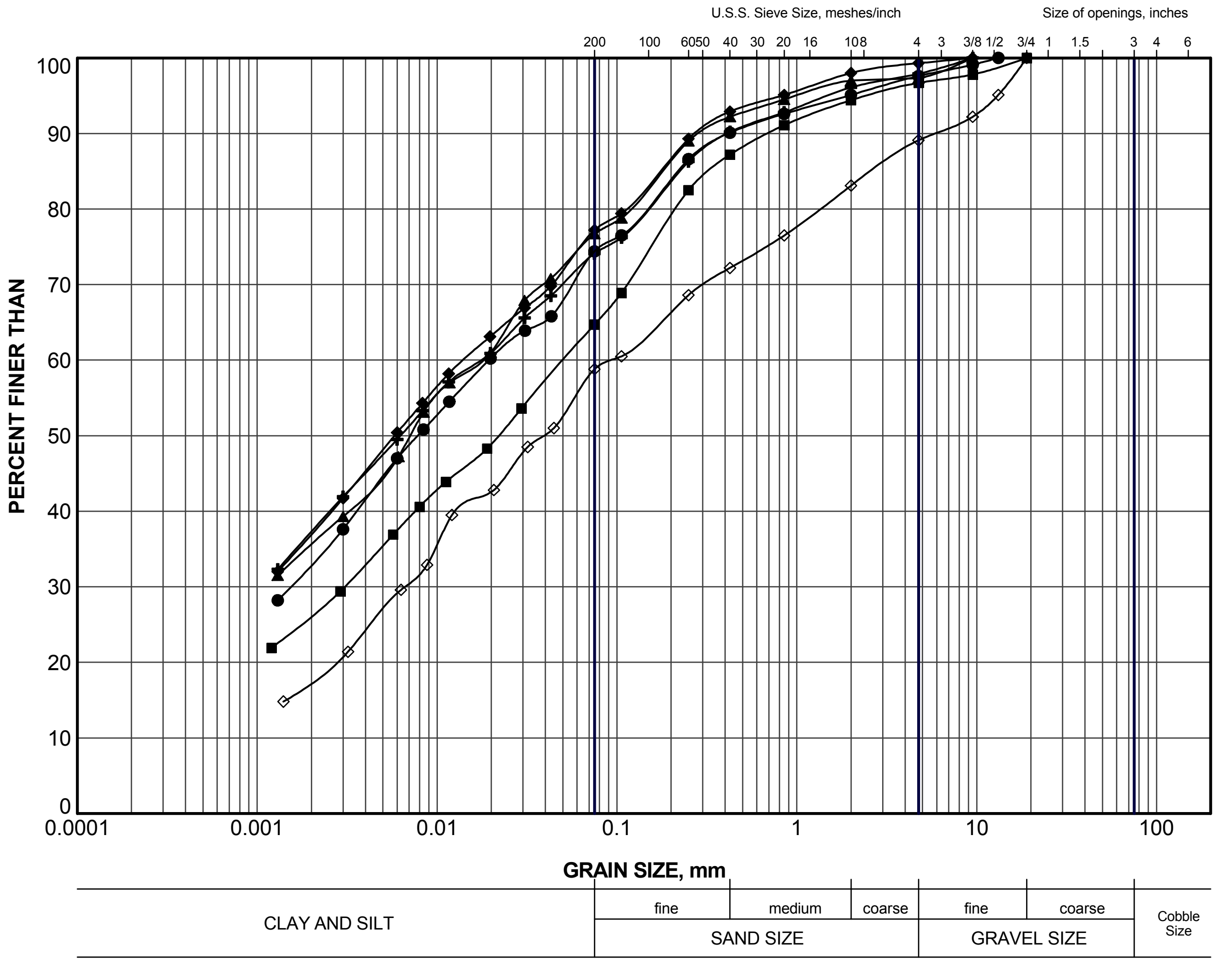
Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-180-0-3		FILE No.	0611301800-3-D020B6
			SCALE AS SHOWN	REV. 0
CADD	DCH	Feb. 08/08		
CHECK			**FIGURE B-6**	

DRAWING FILE: 0611301800-3-D020B6.DWG Plot Date: Feb 11, 2008 - 3:04pm

APPENDIX C

ESSEX COUNTY ROAD 22 AND LESPERANCE ROAD (SITE C)
LABORATORY TEST DATA



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	103	5	177.6
■	103	11	173.8
▲	103	18	163.0
✚	103	22	158.5
◆	103	27	150.9
◇	103	32	143.3

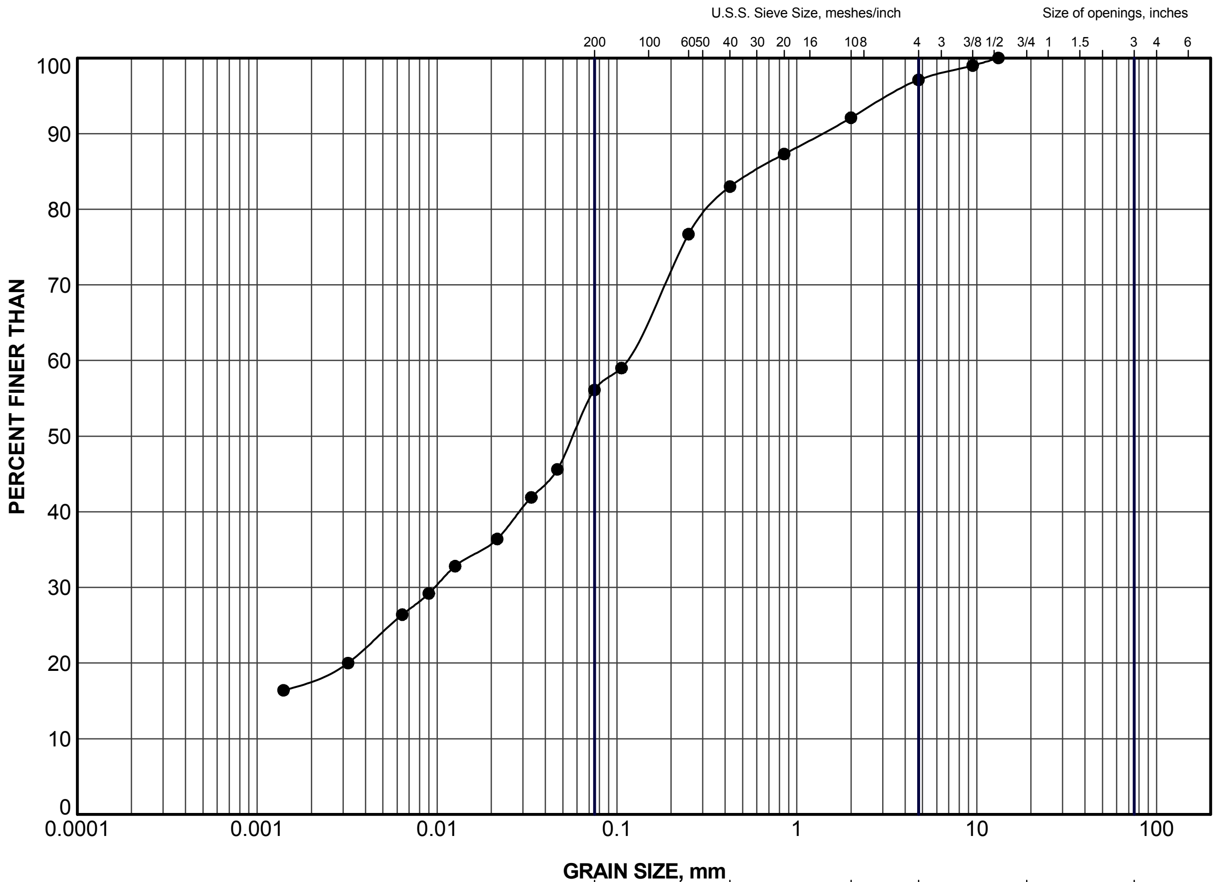
PROJECT
ESSEX COUNTY ROAD 22/LESPERANCE ROAD OVERPASS
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

TITLE
GRAIN SIZE DISTRIBUTION
CLAYEY SILT (TILL)

Golder Associates
LONDON, ONTARIO

PROJECT No. 06-1130-180-0-3			FILE No. 0611301800-3-F030C1
			SCALE N/A REV.
DRAWN	DCH	Feb. 08/08	**FIGURE C-1**
CHECK			

LDN_MTO_NEW GLDR_LDN.GDT



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	103	13	170.7

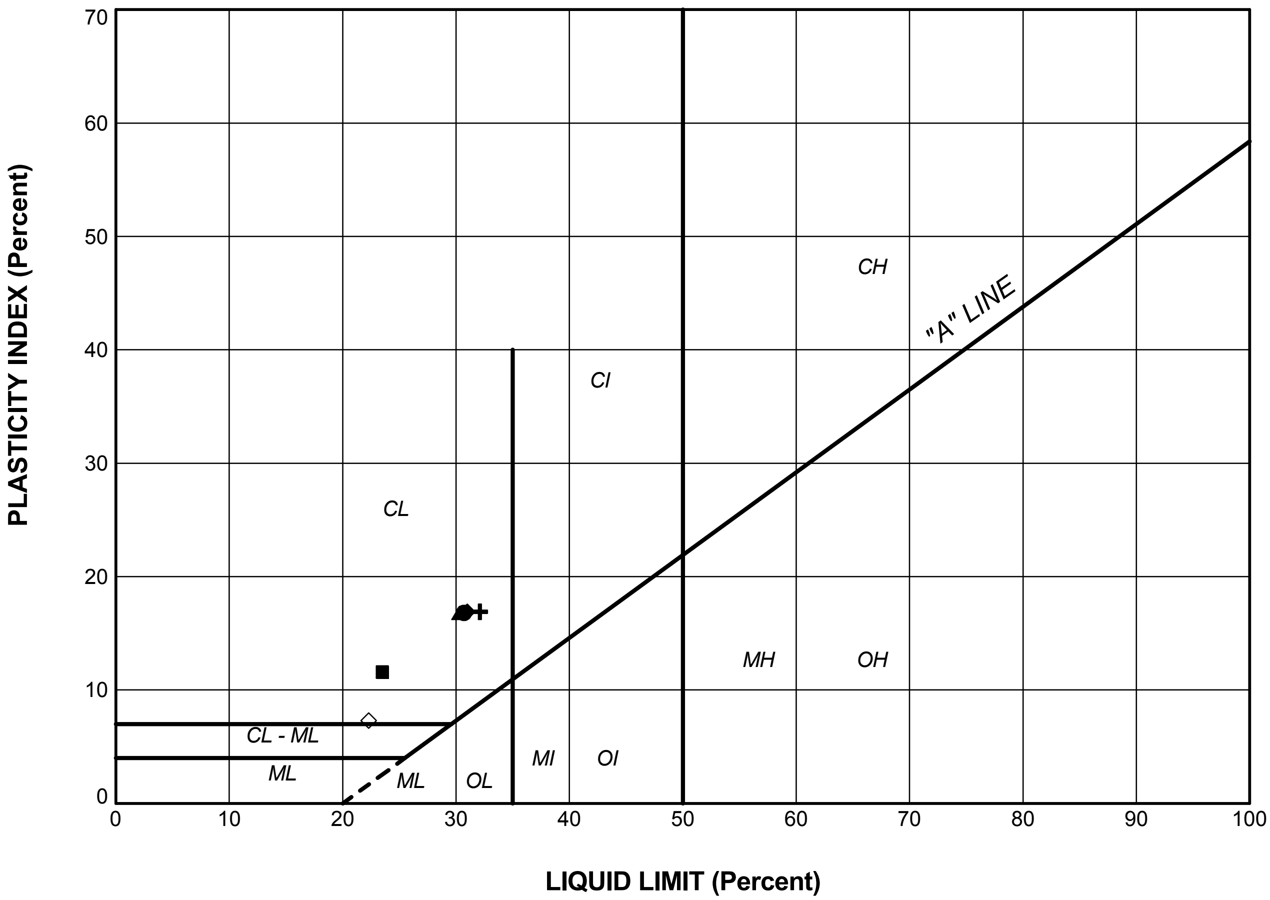
PROJECT
ESSEX COUNTY ROAD 22/LESPERANCE ROAD OVERPASS
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

TITLE
GRAIN SIZE DISTRIBUTION
SANDY SILT (TILL)

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-180-0-5	FILE No.	0611301800-3-F030C2
		SCALE N/A	REV.
DRAWN	BRS	Feb. 08/08	
CHECK			FIGURE C-2

LDN_MTO_NEW GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
CLAYEY SILT (TILL)					
●	103	5	30.7	13.9	16.8
■	103	11	23.5	11.9	11.6
▲	103	18	30.3	13.5	16.8
✚	103	22	32.1	15.2	16.9
◆	103	27	31.0	14.1	16.9
◇	103	32	22.3	15.0	7.3

PROJECT
ESSEX COUNTY ROAD 22/LESPERANCE ROAD OVERPASS
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

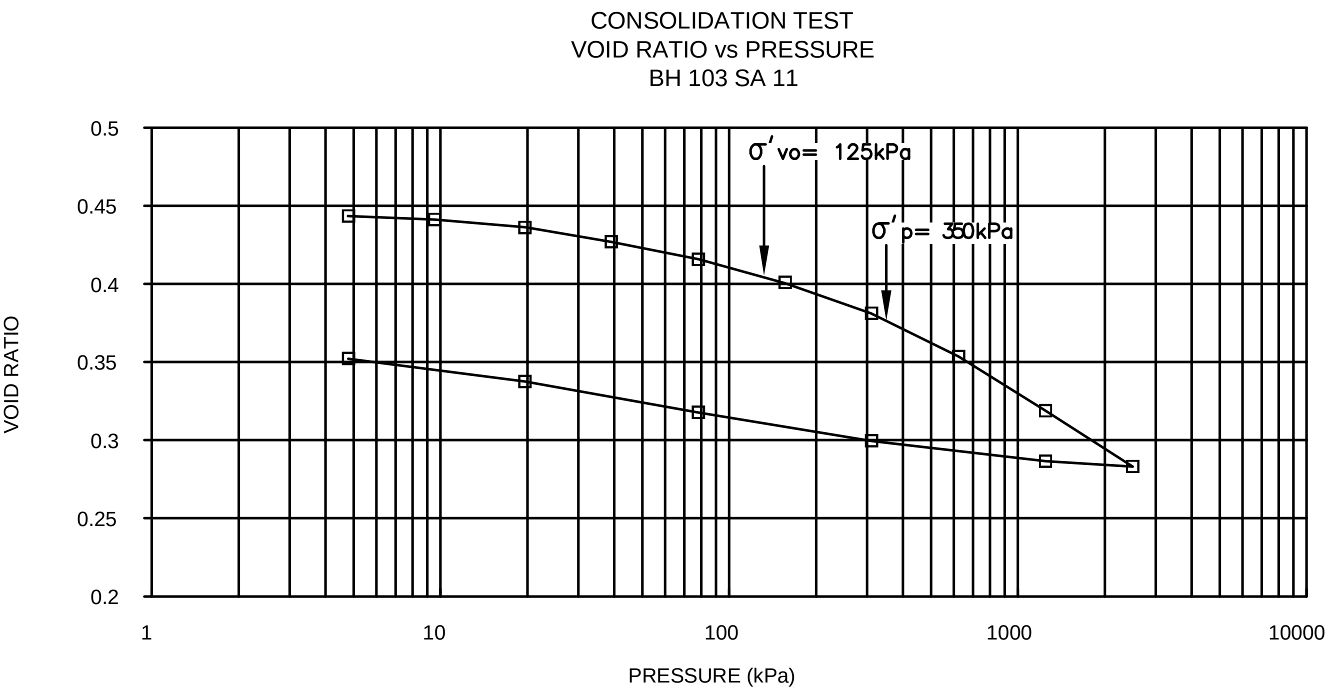
TITLE

PLASTICITY CHART

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-180-0-3		FILE No. 0611301800-3-F030C3	
			SCALE N/A	REV.
DRAWN	DCH	Feb. 08/08	**FIGURE C-3**	
CHECK				

MTO_PI GLDR_LDN.GDT



PROJECT

GRADE SEPERATION AT ESSEX COUNTY ROAD 22
ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING
G.W.P. 3031-06-00

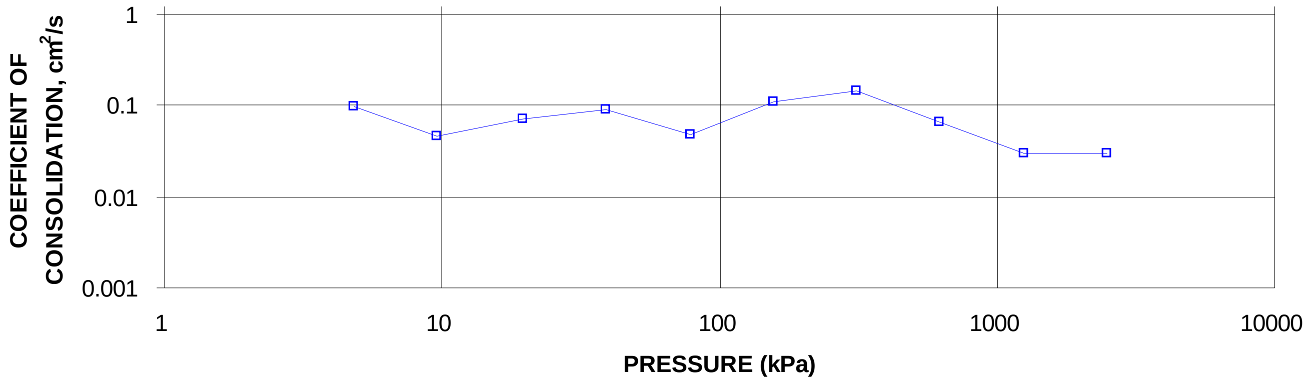
TITLE

CONSOLIDATION TEST

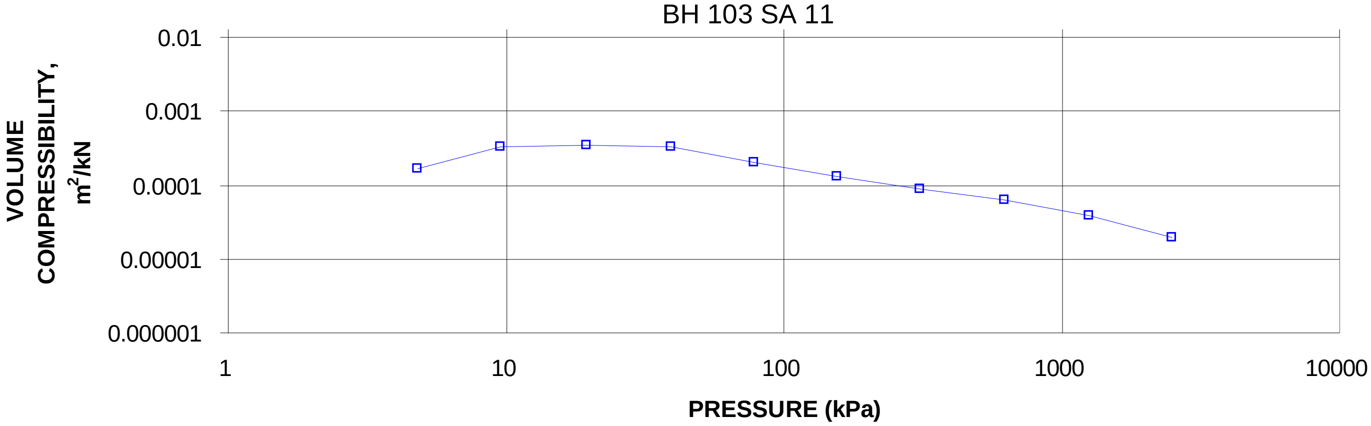
Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-180-0-3		FILE No.	0611301800-3-D030C4
			SCALE AS SHOWN	REV. 0
CADD	DCH	Feb. 08/08	FIGURE C-4	
CHECK				

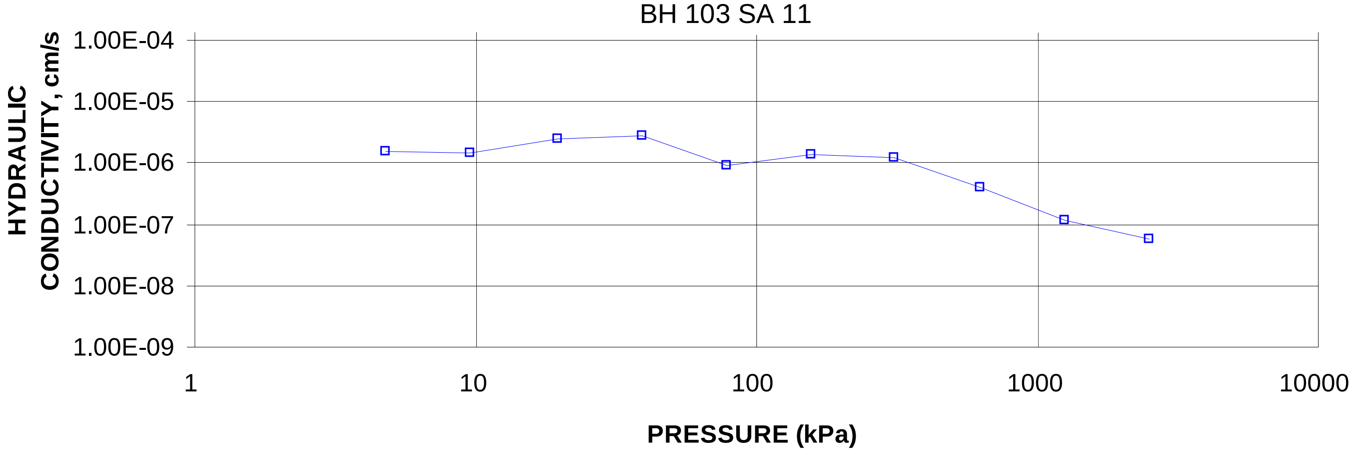
CONSOLIDATION TEST
CV cm²/s vs PRESSURE (kPa)
BH 103 SA 11



CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH 103 SA 11



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 103 SA 11



PROJECT	COUNTY ROAD 22/LESPERANCE ROAD OVERPASS ESSEX COUNTY ROAD 19 (MANNING ROAD) WIDENING G.W.P. 3031-06-00
TITLE	**OEDOMETER CONSOLIDATION SUMMARY**

Golder Associates
LONDON, ONTARIO

PROJECT No.	06-1130-180-0-3		FILE No.	0611301800-3-D030C5
			SCALE	AS SHOWN
CADD	DCH	Feb. 08/08	REV.	0
CHECK				

FIGURE C-5

DRAWING FILE: 0611301800-3-D030C5.DWG Plot Date: Feb 11, 2008 - 3:06pm

APPENDIX D

SITE PHOTOGRAPHS

SITE PHOTOGRAPHS

Photo 1: Site A – CPR and Essex County Road 19.

Photo 2: Site A – CPR and Essex County Road 19.

SITE PHOTOGRAPHS

Photo 3: Site B – Essex County Road 19 and Essex County Road 22.

Photo 4: Site B – Essex County Road 19 and Essex County Road 22.

SITE PHOTOGRAPHS

Photo 5: Site C – Lesperance Road and Essex County Road 22.

Photo 6: Site C – Lesperance Road and Essex County Road 22.