



August 2010

FOUNDATION INVESTIGATION AND DESIGN REPORT

Westmount Road Overpass (Site No. 33-228)
Widening of Highway 7/8
From 0.9 Km West of Fischer-Hallman Road Interchange
Easterly to 0.8 Km East of Courtland Avenue Interchange
Kitchener
GWP 131-98-00, Purchase Order Number 3007-E-0024
Ministry of Transportation, Ontario - West Region

Submitted to:

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REPORT



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LIST OF ABBREVIATIONS

LIST OF SYMBOLS

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(Geocres Report No. 40P08-031)

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Site Photographs



**FOUNDATION INVESTIGATION AND DESIGN REPORT
WESTMOUNT ROAD OVERPASS - SITE NO. 33-228**

PART A

FOUNDATION INVESTIGATION REPORT

**WESTMOUNT ROAD OVERPASS (SITE NO. 33-228)
WIDENING OF HIGHWAY 7/8
FROM 0.9 KM WEST OF FISCHER-HALLMAN ROAD
INTERCHANGE EASTERLY TO 0.8 KM EAST OF
COURTLAND AVENUE INTERCHANGE, KITCHENER
GWP 131-98-00, PURCHASE ORDER NUMBER 3007-E-0024
MINISTRY OF TRANSPORTATION - WEST REGION**



FOUNDATION INVESTIGATION AND DESIGN REPORT WESTMOUNT ROAD OVERPASS - SITE NO. 33-228

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 detail design work for GWP 131-98-00. The project involves the detail design for the widening of Highway 7/8 (Conestoga Parkway) in Kitchener, Ontario.

This report addresses the proposed widening and rehabilitation of the twin Highway 7/8 overpass structures at Westmount Road (Sites 33-228-W and 33-228-E).

The purpose of the foundation investigation is to determine the subsurface conditions at the location of the proposed structure replacement 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 P81-3002 dated April 8, 2008 and our letters dated July 21 and 22, 2008. The work was carried out in accordance with our Quality Control Plan for Foundation Engineering dated July 4, 2008.

Dillon provided Golder Associates with preliminary drawings for this project in digital format.



2.0 SITE DESCRIPTION

The Highway 7/8 – Westmount Road Overpass is located in the south-central area of Kitchener, Ontario. The site is situated between the Fischer-Hallman Road interchange to the west and the Homer Watson Boulevard Interchange to the east. The location of the project is shown on the Key Plan, Figure 1.

The existing overpass consists of twin structures, each with three spans. This section of Highway 7/8 is currently a four lane divided highway with two lanes in each direction. In the overpass area, the westbound and eastbound lanes are separated by a low concrete median. Beyond the overpass structure, the lanes are separated with a depressed grassed median. The highway is oriented generally in an east-west direction. Westmount Road East is a four lane urban arterial roadway which runs north-south in the vicinity of Highway 7/8. At the time the structure was erected, Westmount Road was known as Filsinger Road.

Original grades in the area of the structure varied from elevation 330.0 metres at the east abutment to 333.5 metres at the west abutment. Adjacent land use is typically urban residential.

Site Photographs are provided in Appendix C.

2.1 Site Geology

This project lies within the physiographic region of southwestern Ontario known as the Waterloo Hills¹. The soils generally consist of sandy hills, some consist of sandy till while others are kames or kame moraines, with outwash sands deposited in the valleys. Adjoining the sandy hills is the Grand River spillway system comprised of alluvial terraces of sand and gravel.

Based on the Ministry of Northern Development and Mines Map P.2559 entitled “Quaternary Geology, Stratford Area”, the site lies in an area of primarily ice contact sands deposited in the Pleistocene era. Adjacent to the site, the Maryhill clayey till is indicated.

The Geologic Survey of Canada Map 1263A entitled “Geology, Toronto-Windsor Area, Ontario” indicates that the subcropping bedrock in the area of site is dolomite and mudstone of the Salina formation of Upper Silurian age. Based on the Ontario Department of Mines Preliminary Map No. P.168 entitled “Bedrock Topography Series, Stratford Sheet”, bedrock at the site is at about elevation 260 metres or some 75 metres below ground surface.

¹ L.J. Chapman and D.F. Putnam: The Physiography of Southern Ontario, Third Edition. Ontario Geological Survey, Special Volume 2, 1984.



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3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between October 8 and November 24, 2008, and June 2 to June 3, 2009 during which time ten boreholes were drilled at the locations shown on the Borehole Location Plan, Drawing 1. The table below summarizes the borehole locations, ground surface elevations at the borehole locations and borehole depths.

Borehole	Location (m)		Ground Surface Elevation	Borehole Depth
	Northing	Easting	(m)	(m)
301	4 809 768	223 607	333.67	19.60
302	4 809 734	223 623	333.52	27.89
303	4 809 728	223 602	333.17	20.27
304	4 809 762	223 586	333.46	14.17
305	4 809 724	223 575	339.23	8.08
306	4 809 726	223 583	339.17	18.75
307	4 809 744	223 644	338.45	15.70
308	4 809 769	223 627	338.55	23.27
309	4 809 772	223 638	338.36	19.51
310	4 809 748	223 561	339.27	14.17

The investigation was carried out using truck mounted CME 75 power augers supplied and operated by specialist drilling contractors. In the boreholes, samples of the overburden were obtained at 0.75 and 1.5 metre intervals of depth using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures. The boreholes were terminated between 8.1 and 27.9 metres below the exiting pavement or ground surface. Groundwater conditions in the boreholes were observed throughout the drilling operations and a piezometer and standpipe were installed in borehole 301 as indicated on the corresponding Record of Borehole sheet. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 128/03 amended to 372/07.

The field work was monitored on a full-time basis by experienced members of our engineering staff who located the boreholes in the field, monitored the drilling, sampling and in situ testing operations and logged the boreholes. The samples were identified in the field, placed in labelled containers and transported to our London laboratory for further examination and testing. Index and classification tests, consisting of water content determinations, grain size distribution analyses and Atterberg limits determinations, were carried out on selected samples. The results of the testing are shown on the Record of Borehole sheets and in Appendix A. The locations of the boreholes are shown on the Record of Borehole sheets and on Drawing 1, attached.

In addition, the applicable information from the original geotechnical investigation for the overpass structures was incorporated into this report. Data from boreholes 4 and 8 from Geocres Report No. 40P08-031 entitled "Foundation Investigation Report For Filsinger Road Overpass, Kitchener-Waterloo Expressway, District No. 4 (Hamilton), W.J. 67-F-102 – W.P. 628-64" dated December 28, 1967 were used to supplement the current data.



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The Record of Borehole sheets for previous boreholes and associated laboratory test data are presented in Appendix B. The table below summarizes the locations, ground surface elevations and depths of the previous boreholes:

Borehole	Location (m)		Ground Surface Elevation	Borehole Depth
	Northing	Easting	(m)	(m)
4	4 809 753	223 572	332.99	12.65
8	4 809 738	223 634	330.04	12.53

The locations of the previous boreholes are shown in plan on Drawing 1 and are noted on the Record of Borehole sheets. The locations of the previous boreholes should be considered approximate since the locations were referenced to imperial chainages and offsets rather than metric MTM coordinates.



4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

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 Appendix A. 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.

In summary, the boreholes drilled at the site revealed a highly variable and complex stratigraphy which consisted of existing pavement structure or topsoil overlying fill materials which were generally underlain, in sequence, by sand and sandy silt or clayey silt till and/or clayey silt to silty clay over glacial till. The cohesive deposits are interlayered with silt, sand, silty sand or sand and gravel. The profile along the centreline of Highway 7/8 shown on Drawing 2, has been simplified for clarity. Reference should be made to the Records of Borehole sheets and sections shown on Drawings 2, 3 and 4 for additional details.

The following is a summary of the site stratigraphy.

4.2 Soil Conditions

4.2.1 Pavement Structure

Asphalt was encountered at the pavement surface in boreholes 305, 306, 307, 309 and 310. The asphalt was up to 150 millimetres thick at the borehole locations. An approximately 120 millimetre thick layer of buried asphalt was encountered beneath the sandy silt fill in borehole 301.

Pavement granulars (sand and gravel) were encountered beneath the asphalt in boreholes 301, 305, 306, 307, 309 and 310. The granulars were about 0.2 to 1.2 metres thick.

4.2.2 Topsoil and Fill

Layers of topsoil were encountered at the ground surface in boreholes 302, 303 and 304. The topsoil layers were about 80 to 150 millimetres thick with an average thickness of about 125 millimetres. A 60 millimetre thick layer of buried topsoil was encountered at elevation 330.3 metres in borehole 301.

Fill was encountered at ground surface in boreholes 301 and 308, beneath the pavement structure from elevation 331.4 to 338.9 metres in boreholes 301, 305, 306, 307, 309 and 310 and below the topsoil in boreholes 302, 303 and 304 from elevation 333.0 to 333.4 metres. The fill was variable and consisted of clayey silt materials below the highway pavement structure and silty sands and sand adjacent to Westmount Road. The fill materials ranged in thickness from 1.1 to 7.6 metres.

A hydrocarbon odour was noted in the fill in borehole 304. An Environmental Team from Dillon is currently investigating potential hydrocarbon impacts at this site. Golder is assisting by advancing the boreholes and sampling. The results of the environmental investigation will be presented in a separate report prepared by Dillon.



The fill had N values, as determined in the standard penetration testing, of 3 to 49 blows per 0.3 metres. Samples of the fill had in situ water contents of 10 to 30 per cent. Samples of the cohesive fill materials had average plastic and liquid limits of 14 and 22 per cent, respectively, and a plasticity index of 8 per cent based on five Atterberg limits determinations. These data are provided on the Plasticity Chart, Figure A-10.

Grain size distribution curves for samples of the fill recovered from the standard penetration testing are provided on Figure A-1.

4.2.3 Clayey Silt

Clayey silt was encountered beneath the buried topsoil in borehole 301, beneath the fill in boreholes 303, 304 and 310, beneath the sandy silt in boreholes 304 and 305, beneath the silty fine sand in borehole 307 and at the ground surface of borehole 4 (40P08-031). For the purposes of this report, the materials described as clayey silt to silt and clayey silt to silty clay in borehole 8 (40P08-031) are considered to be clayey silt. Where fully penetrated, the clayey silt was encountered between about elevation 316.6 and 334.8 metres and was about 0.4 to 6.3 metres thick at the borehole locations with an average thickness of about 2.4 metres.

The clayey silt had N values of 8 to greater than 100 blows per 0.3 metres indicate a firm to hard consistency. The clayey silt was of low plasticity based on seven Atterberg limits determinations with liquid limits of 22 to 34 per cent and plasticity indices of 8 to 17 per cent. These data are provided on the Plasticity Chart, Figure A-10. The natural water content ranged from 17 to about 22 per cent.

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

4.2.4 Sandy Silt

Sandy silt was encountered beneath the fill in boreholes 305, 306, 307 and 308, beneath the clayey silt in boreholes 302, 304, 305, 309, 310 and borehole 4 (40P08-031). The sandy silt was encountered between about elevation 318.7 and 336.3 metres. The sandy silt layers were 0.6 to 3.5 metres thick at the borehole locations with an average thickness of about 2.0 metres.

The sandy silt had N values of 13 to greater than 100 blows per 0.3 metres indicating a compact to very dense relative density with natural water contents ranging from 14 to 18 per cent. It should be noted that peaty pockets and/or layers were encountered in the sandy silt in borehole 307 between elevations 328.4 and 329.5 metres. A sample of the sandy silt from this layer had a water content of about 40 per cent.

Grain size distribution curves for samples of the sandy silt recovered from the standard penetration testing are provided on Figure A-2.

4.2.5 Clayey Silt Till

Clayey silt till was encountered beneath the fill in borehole 302, beneath a clayey silt layer in borehole 301, beneath the fill in borehole 302, beneath the sandy silt in borehole 305 and beneath the silt in borehole 308. The clayey silt till layers were intercepted from elevations 308.2 to 332.2 metres. Boreholes 302 and 305 were terminated in the clayey silt till after exploring it for about 2.6 and 1.1 metres, respectively. Where fully penetrated, the clayey silt till layers were about 1.5 to 7.6 metres thick with an average thickness of about 4.3



metres. Although not specifically encountered in the boreholes, cobbles and boulders should be anticipated within the clayey silt till due to the depositional history of glacial till materials.

The clayey silt till had N values of 14 to greater than 100 blows per 0.3 metres indicating a stiff to hard consistency. The clayey silt till was of low plasticity based on three Atterberg limits determinations with liquid limits of 17 and 27 per cent, and plasticity indices of 6 to 13 per cent. These data are provided on the Plasticity Chart, Figure A-10. The natural water contents ranged from about 10 to 17 per cent that is at or below the plastic limit.

Grain size distribution curves for samples of the clayey silt till recovered from the standard penetration testing are provided on Figure A-4.

4.2.6 Sand

Sand was encountered beneath the sandy silt in boreholes 301, 302, 304, 306, 310 and 4 (40P08-031), beneath the silty fine sand in borehole 303, beneath the clayey silt in borehole 307 at the ground surface of borehole 8 (40P08-031). These layers were encountered between about elevation 317.0 and 329.1 metres. Boreholes 304, 307 and 4 (40P08-031) were terminated in the sands after exploring them for about 1.1 to 5.6 metres. Where fully penetrated, the sands were about 1.5 to 3.2 metres thick. The sands had N values of 10 to over 100 blows per 0.3 metres indicating a compact to very dense relative density. The natural water contents were about 12 to 21 per cent.

Grain size distribution curves for samples of the sand recovered from the standard penetration testing are provided on Figure A-5.

4.2.7 Silty Fine Sand

Silty fine sand was encountered beneath the fill in borehole 309, clayey silt till and silt in borehole 302, beneath the clayey silt in borehole 303 and beneath the sandy silt in boreholes 307 and 308 and beneath the sand and gravel in borehole 308. The silty fine sand was encountered between about elevation 317.8 and 331.7 metres. Borehole 308 was terminated in silty fine sand after exploring it for about 2.5 metres. Where fully penetrated, the silty fine sand layers were 0.8 to 4.0 metres thick.

The silty fine sand had N values of 2 to greater than 100 blows per 0.3 metres indicating a very loose to very dense relative density with natural water contents from 11 to 22 per cent.

Grain size distribution curves for samples of the silty fine sand recovered from the standard penetration testing are provided on Figure A-6.

4.2.8 Organic Silt

The silty fine sand in borehole 309 was underlain by a 1.3 metre thick layer of organic silt at elevation 328.6 metres. The organic silt was soft to very stiff with N values of 2 and 19 blows per 0.3 metres at the contact with the underlying compact sandy silt. Water contents of 32 and 52 per cent were measured on samples of the organic silt.



4.2.9 Silt

Layers of compact silt were encountered beneath the silty fine sand in boreholes 302 and 308 and beneath the silty clay in borehole 302. The silts were encountered between about elevation 323.3 and 328.5 metres. The silt layers were about 1.4 to 1.7 metres thick and had N values of 15 to 21 blows per 0.3 metres with a natural water content of about 15 per cent.

A grain size distribution curve for a sample of the silt recovered from the standard penetration testing in borehole 302 is provided on Figure A-7.

Peaty layers and/or pockets were encountered in the silt in borehole 308 between about elevations 327.0 and 328.5 metres.

4.2.10 Silty Sand and Gravel to Sand and Gravel

Layers of very dense silty sand and gravel were encountered beneath the sandy silt till in borehole 302 at elevation 309.6 metres and beneath the sandy silt in borehole 306 at elevation 333.2 metres. The silty sand and gravel was 1.4 and 1.1 metres thick in boreholes 302 and 306, respectively. The silty sand and gravel had N values greater than 100 blows per 0.3 metres.

Layers of very dense sand and gravel were encountered beneath the clayey silt in borehole 301 at elevation 321.2 metres and beneath the clayey silt till in borehole 308 at elevation 319.4 metres. The sand and gravel layers were 0.8 and 1.6 metres thick in boreholes 301 and 308, respectively. The sand and gravel had N values of 84 to 109 blows per 0.3 metres. A sample of the sand and gravel from borehole 308 had a water content of about 10 per cent.

A grain size distribution curve for a sample of the sand and gravel recovered from the standard penetration testing in borehole 308 is provided on Figure A-8.

4.2.11 Silty Clay

Layers of very stiff to hard silty clay were encountered beneath the sand in borehole 306, beneath the lower silty fine sand layers in boreholes 302 and 303 and beneath the clayey silt in borehole 306. These layers were encountered between about elevation 315.5 and 327.6 metres. Boreholes 303 and 306 were terminated in the silty clay after exploring it for about 1.1 to 2.6 metres. Where fully penetrated, the silty clay layers were about 0.7 to 3.1 metres thick with an average thickness of about 2.2 metres. The silty clay had N values of 25 to greater than 100 blows per 0.3 metres with a natural water content of about 16 per cent.

4.2.12 Sandy Silt Till

Very dense sandy silt till was encountered at about elevation 312.8 metres beneath the clayey silt till in borehole 302.

The sandy silt till was about 3.2 metres thick at the borehole location and had N values of 65 to greater than 100 blows per 0.3 metres with a natural water content of about 10 per cent. The sandy silt till is a borderline silt to clayey silt of low plasticity with corresponding plastic and liquid limits of 10 and 16 per cent, respectively, and a



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plasticity index of 6 per cent based on a single Atterberg limits determination. These data are provided on the Plasticity Chart, Figure A-10.

A grain size distribution curve for a sample of the sandy silt till recovered from the standard penetration testing in borehole 302 is provided on Figure A-9. Although not specifically encountered in the borehole, cobbles and boulders should be anticipated within the sandy silt till due to the depositional history of glacial till materials.

4.3 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling and piezometers and standpipes were installed in borehole 301. Installation details are provided on Record of Borehole 301 following the text of this report. A summary of the encountered and measured groundwater levels is provided in the following table:

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)	Installation	Measured Groundwater Elevation (m)		
				October 9, 2008	August 15, 2009	June 3, 2010
301	333.67	321.2	Standpipe Deep Piezometer	330.93 329.45	331.18 330.67	331.16 330.39
302	333.52	330.5 318.3	-	-	-	-
303	333.17	317.9	-	-	-	-
304	333.46	Dry	-	-	-	-
305	339.23	Dry	-	-	-	-
306	339.17	330.0	-	-	-	-
307	338.45	327.8	-	-	-	-
308	338.55	329.4	-	-	-	-
309	338.36	329.3 320.7	-	-	-	-
310	339.27	327.7	-	-	-	-
4 (40P08-031)	332.99	331.8	-	-	-	-
8 (40P08-031)	330.04	329.5	-	-	-	-

Boreholes 304 and 305 remained dry during drilling. Groundwater was encountered in the remaining boreholes at depths of 3.0 to 17.7 metres or between elevation 320.7 and 331.8 metres.

A standpipe and a piezometer were installed in borehole 301. A shallow standpipe was installed within the fill and a deep piezometer was installed within the lower sands. On June 3, 2010, the water level in the shallow piezometer was about 2.5 metres below ground surface or at about elevation 331.2 metres. The water level in the deep piezometer was about 3.3 metres below ground surface or at about elevation 330.4 metres.

Groundwater was encountered in previous boreholes 4 and 8 (40P08-031) at elevations 331.8 and 329.5 metres respectively or 1.2 and 0.5 metres, respectively, below the ground surface.



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Based on the measured and encountered water levels, and the change in soil colour, the inferred groundwater level is at elevation 331 metres. The groundwater levels are expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring melt conditions.



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5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by All Terrain Drilling Ltd. and Aardvark Drilling Inc., both of which are Ontario Ministry of Environment licensed well contractors. The field operations were supervised by Mr. Michael Arthur and Mr. Dan Babcock, P.Eng. under the direction of Mr. David J. Mitchell. The 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. This report was prepared by Mr. Michael E. Beadle, P.Eng. and Ms. Dirka U. Prout, P.Eng. under the direction of the Team Leader, 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.

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**FOUNDATION INVESTIGATION AND DESIGN REPORT
WESTMOUNT ROAD OVERPASS - SITE NO. 33-228**

PART B

FOUNDATION DESIGN REPORT

**WESTMOUNT ROAD OVERPASS (SITE NO. 33-228)
WIDENING OF HIGHWAY 7/8
FROM 0.9 KM WEST OF FISCHER-HALLMAN ROAD
INTERCHANGE EASTERLY TO 0.8 KM EAST OF
COURTLAND AVENUE INTERCHANGE, KITCHENER
GWP 131-98-00, PURCHASE ORDER NUMBER 3007-E-0024
MINISTRY OF TRANSPORTATION - WEST REGION**



6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides our recommendations on the foundation aspects of the design of the proposed symmetrical widening of the twin Westmount Road overpasses and associated embankment widenings based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The overpass consists of two separate three span concrete bridges which will be rehabilitated and widened approximately 2 metres on each side.

The subsoils encountered in the boreholes put down during the investigation consist of a highly variable and complex stratigraphy which consisted of existing pavement structure or topsoil overlying fill materials which were generally underlain, in sequence, by sand and sandy silt or clayey silt till and/or clayey silt to silty clay over glacial till. The cohesive deposits are interlayered with silt, sand, silty sand or sand and gravel.

The inferred groundwater level is approximately elevation 331 metres.

6.2 Existing Structure

The Westmount Road Overpass structures were erected in 1971. Department of Highways, Ontario (DHO) Drawing No. D-6345-1 entitled "General Arrangement: Kitchener-Waterloo Expressway, Filsinger Road Overpass" dated March 1962, DHO Drawing No. D-6345-3 entitled "Footing and Pile Layout & Details" dated March 1962 and Geocres Report No. 40P08-031. As noted previously, Westmount Road at this location was primarily known as Filsinger Road. The overpass structure is comprised of twin three span concrete structures with post tensioned decks. The overall structure is approximately 49.1 metres long and 36.3 metres wide.

The original design information indicates that the bridge piers and abutments were to be supported on concrete filled nominal 324 millimetre diameter steel tube piles with a 6.4 millimetre diameter wall thickness. Design DHO Drawing No. D-6345-3 indicated variable pile lengths ranging from 12.2 to 19.2 metres. Based on this information, the piles were founded at approximate elevation 317.2 metres at the west abutment, elevation 316.8 metres at the west pier, elevation 320.1 metres at the east pier and elevation 319.6 metres at the east abutment. The geotechnical report for this site recommended a working stress design load of 534 kilonewtons with piles driven to approximate tip elevations of 316.76 to 320.12 metres.

The elevation of the Highway 7/8 pavement surface at the abutments is approximately at 339 metres. The existing embankments are up to 5.5 metres high.



6.2.1 Geotechnical Resistances for Existing Foundations

The design drawings provided indicate that the existing structures were to be supported on 324 millimetre diameter concrete filled steel tube piles driven to the very dense granular or very stiff to hard cohesive deposits at about elevation 317 to 320 metres with a working stress design capacity of 60 tonnes (534 kilonewtons(kN)) per pile. Recent analysis of the piles by Golder using Limit States Design suggested that the factored geotechnical axial resistance of the piles was 900 kN at Ultimate Limit States (ULS) and a geotechnical resistance of 600 kN at Serviceability Limit States (SLS).

The following factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical resistances at Serviceability Limit States (SLS) can be utilized for a structural assessment of the existing structures:

Location	Foundation Type	Cut-Off Elevation	Tip Elevation	Pile Length	Founding Strata	Geotechnical Resistances	
						Factored ULS	SLS
		(m)	(m)	(m)		(kN)	(kN)
West abutment	Tube Pile	336.42	317.22	19.20	Very dense sandy silt/sand to silty sand	900	600
West pier	Tube Pile	332.00	316.76	15.24	Very dense sand	900	600
East pier	Tube Pile	332.31	320.12	12.19	Very dense sandy silt/very stiff to hard clayey silt	900	600
East abutment	Tube Pile	336.04	319.58	16.46	Stiff to hard clayey silt till	900	600

It should be noted that the pile tip and underside of footing elevations as well as the pile lengths are based only on the design drawings as no construction records or as-built information was available.

For the purposes of structural analysis, the following lateral resistances pile at ULS and SLS can be used:

Location	Factored ULS (kN)	SLS (kN)
West Abutment	80	40
West Pier	85	40
East Pier	80	30
East Abutment	80	30



6.2.2 Conversion to Semi-integral Abutments

The existing conventional abutments will be converted to semi-integral abutments. The configuration of the piled abutment foundation features two rows of steel tube piles. At the abutments, the outer row of piles was designed to be spaced at approximately 3.51 metres and the inner row of piles spaced at 2.46 metres. There are also two rows of piles at the piers, with each row of piles spaced 1.32 metres apart. All piles were to be battered outward at 1 horizontal to 6 vertical. The existing foundations and sub-surface conditions are considered to be compatible with a semi-integral abutment design.

6.3 Proposed Work

The existing decks are to be removed, girders installed and new decks constructed with a grade raise of 0.6 to 0.9 metres. In addition, the bridge will be widened symmetrically by about 2 metres. The construction of the widened embankments, grade raises for the existing embankments, widened bridge sections and replacement of the bridge decks will be carried out in stages.

6.4 Bridge Widening Foundations

Fill, in addition to the embankment fills, was placed in the area between the west pier and the east abutment. It is estimated that the fill extends up to approximately 1.5 to 4.4 metres below the surface of Westmount Road which is at approximate elevation 333 metres. Shallow foundations are unsuitable for support of the widened piers and abutments since deep excavations extending to about elevation 329 metres or up to 4.5 metres below Westmount Road and some 2 metres below the groundwater level would be required to reach competent native soils. Due to the presence of moderately compressible cohesive deposits and organic soils near elevation 329 metres in the vicinity of the east pier, deeper excavations and lower founding levels would be required in this area. For these reasons, deep foundations are the preferred technical solution for the foundations of the widened rehabilitated/structure.

The various foundation options considered for this site are compared in Table I. This table includes estimated foundation costs and summaries of the feasibility of each option. The costs given are rough estimates presented to give an order of magnitude cost comparison between alternatives rather than absolute figures.

6.4.1 Deep Foundations

Steel H-piles HP 310 x 110 or concrete filled tube piles 324 millimetres outer diameter (O.D.) with 9.5 millimetre wall thickness driven close ended to the very dense granular soils and hard clayey silt are considered suitable for support of the widened piers and abutments. As noted with piles driven for the existing structure, the pile lengths will be variable.

Geotechnical Axial Resistance – Driven Steel H-Piles

HP 310 x 110 piles can be driven to refusal at or below the elevations shown in the table below using the appropriate factored ULS and unfactored SLS geotechnical resistances. The SLS values assume 25 millimetres of settlement. Flanges should be reinforced as per Ontario Provincial Standard Drawing (OPSD) 3000.100. The



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steel H-piles should be installed and monitored in accordance with SP903S01 and OPSD 3000.150. The H-piles should be equipped with reinforced flanges and a Type I driving shoe as per OPSD 3000.100. In accordance with Special Provision 903S01, piles driven into granular soils should be re-tapped to confirm the set after adjacent piles have been driven.

A pile note is to be added to the foundation drawing stating that piles are to be driven in accordance with Standard SS 103-11 using a maximum ultimate resistance of two times the factored ULS value shown in the table below and must be driven below the elevations shown in the above table. The wording of the pile note should match Note 2 of Section 3.3.3 of the MTO Structural Manual.

It is assumed that the cut-off elevations will be similar to those used for the existing structure since the underside of pile cap elevations for the widened portions of the overpass structures will match those of the existing pile caps.

Location	Cut-Off Elevation	Tip Elevation	Approximate Pile Length	Founding Strata	Geotechnical Resistances	
					Factored ULS	SLS
	(m)	(m)	(m)		(kN)	(kN)
West abutment	336.4					
- north side		321	15.4	Very dense sand	1500	1000
- south side		324	15.4	Hard clayey silt	1500	1000
West pier	332.0					
- north side		321	11.0	Very dense sand	1500	1000
- south side		315	17.0	Hard silty clay	1500	1000
East pier	332.3					
- north side		318	14.3	Very dense sand	1500	1000
- south side		309	23.3	Very dense silty sand and gravel	1500	1000
East abutment	336.0					
- north side		318	18.0	Very dense sand & gravel	1500	1000
- south side		318	18.0	Hard clayey silt	1500	1000



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Geotechnical Axial Resistance – Driven Steel Tube Piles

Concrete filled steel tube piles 324 millimetres O.D. with 9.5 millimetre wall thickness driven closed ended may be used for support of the widened abutments and piers. Assuming the tube piles are driven to an appropriate set at about the elevations shown in the table below, the noted factored ULS and unfactored SLS geotechnical resistances can be used for design. The geotechnical resistances have been selected to provide deformation characteristics similar to the existing pile foundations. Higher resistances could be developed by driving the piles deeper but this would result in differential performance between the new and the existing piles. The SLS values assume 25 millimetres of settlement. The steel tube piles should be installed and monitored in accordance with SP903S01 and OPSD 3001.150. In accordance with Special Provision 903S01, piles driven into granular soils should be re-tapped to confirm the set after adjacent piles have been driven. The tube piles should be equipped with a Type I driving shoe as per OPSD 3001.100.

A pile note is to be added to the foundation drawing stating that piles are to be driven in accordance with Standard SS 103-11 using a maximum ultimate resistance of two times the factored ULS value shown in the table below and must be driven below the elevations shown in the above table. The wording of the pile note should match Note 2 of Section 3.3.3 of the MTO Structural Manual.

It was assumed that the cut-off elevations will be similar to those used for the existing structure since the underside of pile cap elevations for the widened portions of the overpass structures will match those of the existing pile caps.

Location	Cut-Off Elevation (m)	Tip Elevation (m)	Approximate Pile Length (m)	Founding Strata	Geotechnical Resistances	
					Factored ULS (kN)	SLS (kN)
West abutment	336.4					
- north side		325	11.4	Very dense sand	900	600
- south side		326	10.4	Hard silty clay	900	600
West pier	332.0					
- north side		322	10.0	Very dense sandy silt	900	600
- south side		323	9.0	Very dense silty sand	900	600
East pier	332.3					
- north side		320	12.3	Very dense sand	900	600
- south side		320	12.3	Very dense sandy silt	900	600
East abutment	336.0					
- north side		322	14.0	Very stiff to hard clayey silt till	900	600
- south side		321	15.0	Hard clayey silt	900	600



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Construction Considerations

It should be noted that cobbles and boulders may be present in the till soils and may impact pile driving operations.

Downdrag Load (Negative Skin Friction)

The newly widened approach embankments and grade raise less than 1 metre will induce some very minor consolidation settlement of the underlying cohesive deposits. While the consolidation settlement is time-dependent most will occur during the construction period. Post-construction settlement of the clayey deposits is not expected to result in the development of negative skin friction acting on the piles.

However, in order to minimize any settlement induced downdrag loads on the piles, the embankment widenings should be constructed as early as possible and at least one month prior to 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 semi-integral abutments, battered piles should provide the resistance to the lateral loading.

The horizontal reaction to the pile can be estimated using the following equation and ranges in subgrade reaction coefficient where:

$$\begin{aligned}
 k_s &= \text{coefficient of horizontal subgrade reaction (MPa/m)} \\
 &= n_h (z/d) \quad \text{for cohesionless soils} \\
 &= \frac{67S_u}{d} \quad \text{for cohesive soils} \\
 d &= \text{pile width or diameter (m)} \\
 n_h &= \text{constant of horizontal subgrade reaction (MPa/m)} \\
 z &= \text{depth below ground surface grade (m)}
 \end{aligned}$$

Soil Type	Elevation (m)		n_h	S_u
	From	To	(MPa/m)	(MPa)
Stiff to hard clayey silt/clayey silt till (north side)	330	324	-	0.08 – 0.20
Compact to very dense sandy silt/sand and gravel/sand/silty fine sand/silt (south side)	330	322	5 - 10	-
Compact to very dense sandy silt (north side)	324	320	5 – 10	-
Very stiff to hard silty clay/clayey silt (south side)	322	319	-	0.15 – 0.20
Very dense sand (north side)	320	317	10 – 12	-
Very dense sandy silt/sand (south side)	319	315	10 – 12	-
Hard clayey silt (north side)	317	314	-	0.20 – 0.40
Hard clayey silt till/silty clay (south side)	315	313	-	0.20 – 0.40
Very dense sandy silt till	313	310	10 – 12	-
Very dense sand and gravel	310	308	10 – 12	-



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Soil Type	Elevation (m)		n_h	S_u
	From	To	(MPa/m)	(MPa)
Hard clayey silt till	308	306	-	0.20 – 0.50

The nature of the fill materials and depth at this site is quite variable. The existing ground surface elevations at the north end of the west abutment and south end of the east abutment are such that at least 3 to 6 metres of fill has been placed below the cut-off elevations at each location, respectively. Therefore, assuming that the cut-off elevations for the proposed piles will be identical to those of the existing piles, the fill material should not be relied upon for resistance to lateral loads at this site.

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

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

The response of HP 310 x 110 steel piles and steel tube piles, 324 millimetre outside diameter by 9.5 millimetre wall thickness to lateral loading was analysed using L-Pile software to generate p-y curves for each abutment and pier location. Group action was not accounted for in this analysis because the proposed spacing of the piles is not known. Two lateral load cases were analysed: the lateral load required to induce 10 millimetre displacement at the pile head was taken as the SLS load case, while the lateral load required to generate a plastic hinge in the pile was taken as the ULS load case. The table below shows the SLS and ULS geotechnical capacities of the piles under lateral loading.

Location	Lateral Load (kN)			
	HP 310 X 110		324 mm O.D. x 9.5 mm	
	Factored ULS	SLS	Factored ULS	SLS
West Abutment	90	50	100	50
West Pier - north end	100	50	100	50
West Pier - south end	100	50	100	50
East Pier	80	50	110	50
East Abutment	90	50	110	50



Monitoring of Existing Structure

The effects of pile driving for the structure widening on the existing Westmount Road overpasses which will be rehabilitated are expected to be minimal. The piles will be driven through 9 to 17 metres of generally compact to dense sandy silt and sand and very stiff clayey silt. With the exception of isolated layers of very dense granular deposits, hard driving conditions are not expected until the founding strata are reached. The number of piles will be limited due to the relatively small extent of widening.

Potential direct vibration damage is expected to be greatest within a radius of 1 pile length or up to 15 metres from the location of the pile installation.² The closest adjacent structure is situated approximately 33 metres away. Discontinuous layers of very loose saturated silty fine sand and soft organic silt soils were encountered between elevation 328 and 329 in borehole 309 in the northeast quadrant of the structure. Based on the foundation investigation, there are no deposits of loose clean uniform sands below the groundwater level that would be subject to densification and/or settlement during pile driving. No shallow zones of hard/very dense soils were encountered which may adversely impact pile driving. Therefore, considering the limited number of relatively short piles to be installed, the distance to the closest structure and the site stratigraphy, the risk of vibration damage during pile driving is considered to be low. Vibration and settlement monitoring are not considered warranted at this site from a foundation engineering perspective.

6.5 Liquefaction Potential and Seismic Analysis

6.5.1 Seismic Parameters

The site is located in Kitchener, in southwestern Ontario. According to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio, A , applicable to this site is 0.05. The corresponding acceleration related seismic zone, Z_a is 1. The following seismic performance zones are applicable to the proposed structure based on the assigned importance category:

Importance Category	Seismic Performance Zone
Lifeline bridge	2
Emergency Route and other bridges	1

We have been informed by Dillon that the structure is not a designated lifeline bridge, and therefore a rigorous seismic analysis for earthquake loads is not required. However, design forces for restraining elements and bridge support lengths must meet the minimum requirements as outlined in CHBDC Clause 4.4.5.1.

The effects of site conditions on the bridge response are to be included in the determination of the seismic loads. The stratigraphy generally consists of surficial topsoils and fills overlying interlayered deposits of generally stiff to very stiff clayey silt to clayey silt and clayey silt till and compact to very dense sands, silts, sand and gravel and sandy silt till. None of the boreholes advanced for the current or previous investigations at this site encountered bedrock. The available mapping indicates that dolomite and mudstone bedrock of the Salina formation is present at a depth of 75 metres or below approximate elevation 260 metres. Based on the site stratigraphy, the soil profile is categorized as Type II with a seismic site response coefficient, S of 1.2 based on the CHBDC criteria.

² Woods, Richard D. : Dynamic Effects of Pile Installations on Adjacent Structures, National Cooperative Highway Research Program Synthesis of Highway Practice 253. National Academy Press, Washington, D.C., 1997.



6.5.2 Seismic Hazard Assessment

The site location has historically been considered to be in an area of low seismicity, with peak ground acceleration (PGA) values between 0.04 to 0.08g from an earthquake with a 10 per cent probability of exceedance in 50 years. A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the Federal Highway Administration recommended procedures.³ Although granular layers with fines contents less than 15 per cent by mass passing the 0.005 millimetre size are present below the groundwater table, these layers were found to have a normalized N value of greater than 22 blows per 0.3 metres and often greater than 30 blows per 0.3 metres. These deposits also dated to the Pleistocene era. Deposits from the Pleistocene historically have a very low to low susceptibility to liquefaction upon strong ground shaking. The liquefaction potential is considered to be relatively low based on the soil profile type, age of the deposits, the presence of predominantly stiff/dense to very dense soils at the site and the historically low seismicity. Therefore a detailed evaluation of the liquefaction potential of the foundation soils, impact of liquefaction on the bridge foundation and the effect of seismic forces on embankment stability and the bridge abutment and retaining walls is not considered warranted.

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 200 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 SP105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with Ontario Provincial Standard Drawing (OPSD) 3101.150 and 3190.100.
- A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design of the abutment wall in accordance with CHBDC Figure 6.6. Compaction equipment should be used in accordance with SP105S10.
- In accordance with CHBDC Clause C6.9.1, the granular fill may be placed either in a zone with a width equal to at least 1.4 metres behind the back of the stem (Case a from Commentary on CHBDC Figure C6.20) or within the wedge-shaped zone defined by a line drawn at a maximum 1 horizontal to 1 vertical extending up and back from the rear face of the footing (Case b from Commentary on CHBDC Figure C6.20).

³ Federal Highway Administration (FHWA). (1997). "Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles." Geotechnical Engineering Circular No. 3: FHWA-SA-97-076, Washington, D.C.



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- For Case a, the restrained case, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM):

Soil unit weight:	20 kN/m ³	
Coefficients of lateral earth pressure:	Active, K_a	0.33
	At rest, K_o	0.50
	Passive, K_p	3.0

- For Case b, the unrestrained case, 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/m ³	21 kN/m ³
Coefficients of lateral earth pressure:		
	Active, K_a	0.31
	At rest, K_o	0.47

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.
- If semi-integral abutment design allows for movement of the bridge deck ends, passive earth pressure may be used in the geotechnical design of the structure. The movements required to fully mobilized passive pressure or resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize the full passive resistance. A resistance factor equal to 0.5 should be applied to the calculated total passive resistance in accordance with Table 6.1 of the CHDBC.

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 if there is sloping ground at the back of the wall.

6.7 Embankments

The embankments will be widened symmetrically about the centreline by 2 metres on each side. The roadway profile grade will also be raised by approximately 0.6 to 0.9 metres at the bridge location. The existing approach embankments are approximately 55 metres wide at the base and up to 6 metres high with side slopes of 2 horizontal to 1 vertical.



6.7.1 Subgrade Preparation and Embankment Construction

All embankment widening is to be constructed in accordance with SP206S03. All surficial topsoil, organic, loose, soft and/or otherwise deleterious materials should be removed from within the area of the embankment and the exposed subgrade soils should be proofrolled prior to fill placement under the direction of qualified geotechnical personnel.

Except for the top 0.5 metres, where Granular B Type III should be placed, the embankment fills should consist of an approved granular borrow such as SSM or Granular B Type I. Embankment fill materials should be placed in maximum 300 millimetre thick loose lifts and properly benched into the existing embankments in accordance with OPSD 208.010 and compacted. Upon completion of filling to the pavement subgrade level, the embankment sideslopes should be trimmed to a final inclination of two horizontal to one vertical or flatter.

6.7.2 Settlement

Settlement of the proposed embankment widening was modelled using Settle^{3D}, a three-dimensional program for the analysis of consolidation and settlement. The widening was modelled using the proposed dimensions of the widening. An infinite embankment widened symmetrically by two metres with a grade raise of 0.9 metres was assumed for the model. Post-construction settlement criteria recommended by MTO of an allowable settlement of 10 to 25 millimetres within 30 metres of an abutment was used to assess post-construction settlement performance of the modified approach embankments.

Settlements in the order of 10 millimetres or less are expected for the widenings. Noting the relatively low grade raise, limited width of the widening areas and presence of very stiff to hard cohesive soils and compact to dense cohesionless soils near the surface, the resulting settlement is expected to occur mainly during construction and will be complete at the end of the construction period. Post-construction settlements in these areas are expected to be minimal and well within the MTO's settlement criteria.

6.7.3 Stability

A factor of safety against deep seated failure of greater than 1.3 is available for embankments constructed with earth fill materials and founded on the stiff to hard clayey silt to sandy silt subgrade soils at the site.

6.8 Excavations and Temporary Cut Slopes

Excavations will be required to remove unsuitable material from the embankment slopes and foundation areas and for the pile caps at the abutments and piers. The excavations will largely be in surficial fill materials, although they may penetrate the native silty sand, sandy silt and clayey silt at the base of the embankments. The groundwater level is expected to be near elevation 331 metres and will fluctuate with seasonal and climatic variations. The excavations are not expected to penetrate the groundwater level, though if they do, seepage volumes are likely to be low due to the fine grained nature of the soil. If necessary, groundwater control may be effected using properly filtered sumps located outside the foundation areas. 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.

6.8.1 Staging and Temporary Roadway Protection

Widening and raising the grade of the existing approach embankments and conversion of the abutments to semi-integral abutments will require installation of temporary roadway protection systems as the construction will occur in stages. The initial stage (Stage 1) consists of preparatory work on the left side to temporarily extend the roadway platform onto the shoulders to accommodate Stage 2 traffic. Once the temporary widening is complete, eastbound traffic will be diverted onto the north side of Highway 7/8 while the southern embankment is widened and the grade raised and the southern side of the overpass structure rehabilitated and widened (Stage 2). Once Stage 2 is complete, all traffic will be diverted onto the eastbound side and the westbound approach embankments and structure will be widened and rehabilitated (Stage 3).

Temporary roadway systems will be installed in the median and at the north and south ends of the existing abutments in order to permit construction of the widened pile caps, construction of new wingwall segments and to accommodate the proposed grade raise. These systems are to be designed to Performance Level 2 by the Contractor. The limits of the systems are to be determined by the contractor but it is anticipated that the temporary roadway protection system will extend between 150 to 200 metres behind each abutment.

In addition, where space is restricted and will not permit open cuts, temporary roadway protection support systems should be installed to support the sides of the excavation and permit the use of vertical cuts.

The temporary support system could consist of driven steel sheet piles or soldier piles and lagging where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds. Support to the system could be in the form of struts and walers in the case of footing excavations or rakers and anchors in the case of roadway protection.

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

Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter.

Passive distributions will depend on the soil type, the support system and tolerable design deformation(s).

The unfactored triangular earth pressure distribution (p in kN/m^2 ; increasing with depth), can be calculated as follows:

$$\begin{aligned} p &= K_a (\gamma H + q) \\ \text{where } H &= \text{the height of the excavation at any point in metres} \\ K_a &= \text{active coefficient of earth pressure} \\ \gamma &= \text{soil unit weight} \\ q &= \text{surcharge for traffic and other loading} \end{aligned}$$



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For the granular fill and native granular materials, the unfactored rectangular earth pressure distribution (p in kN/m^2 ; constant with depth), can be calculated as follows:

$$p = 0.65 K_a (\gamma H + q)$$

where H = the total height of the excavation

K_a = active coefficient of earth pressure

γ = soil unit weight

q = surcharge for traffic and other loading

For the cohesive fill and native cohesive materials, the unfactored trapezoidal earth pressure distribution (p in kN/m^2 ; varying with depth); can be calculated as follows:

$$p = 0.2\gamma H_T \text{ to } 0.4\gamma H_T$$

where H_T = the total height of the excavation

γ = soil unit weight

q = surcharge for traffic and other loading

The support systems may be designed using the following parameters:

Soil Type	Coefficient of Earth Pressure			Internal Angle of Friction (degrees)	Unit Weight (kN/m^3)
	Active, K_a	At Rest, K_o	Passive, K_p		
Clayey Fill	0.36	0.53	2.8	28	20
Granular Fill	0.33	0.50	3.0	30	21
Sandy Silt – Silty Sand	0.33	0.50	3.0	30	20
Clayey Silt	0.36	0.53	2.8	28	20



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The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficients should be adjusted accordingly. The following coefficients, adjusted for a 2 horizontal to 1 vertical slope behind or in front of the shoring should be used for the design of temporary roadway protection systems installed along the crest of the embankment:

Soil Type	Coefficients of Earth Pressure adjusted for 2 Horizontal: 1 Vertical Slope		
	Active, K_a	At Rest, K_o	Passive, K_p
Clayey Fill	0.63	0.77	1.0
Granular Fill	0.54	0.72	1.1
Sandy Silt – Silty Sand	0.54	0.72	1.1
Clayey Silt	0.63	0.77	1.0



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7.0 MISCELLANEOUS

This report was prepared by Ms. Dirka U. Prout, P.Eng. under the direction of the Team Leader, 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.

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TABLE I

COMPARISON OF FOUNDATION ALTERNATIVES

Site 33-228
Westmount Road Overpass
Highway 7/8 Widening
GWP 131-98-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
Spread footings supported on competent native soils	<ul style="list-style-type: none"> • Not feasible 	<ul style="list-style-type: none"> • Ease of construction 	<ul style="list-style-type: none"> • Requires excavations of 3 metres or more in depth to construct footings and 2 metres below groundwater level. Dewatering would be required to retain competent soils at some footing locations • May provide insufficient capacity for foundation loads • Possibility of differential settlement between widened and pre-existing areas if used for foundations 	<ul style="list-style-type: none"> • Less expensive than deep foundation options 	<ul style="list-style-type: none"> • Differential settlement with pile supported foundations • Loosening of foundation soils when excavating below groundwater level
End bearing concrete filled steel tube piles driven to design capacity	<ul style="list-style-type: none"> • Feasible • Preferred technical option 	<ul style="list-style-type: none"> • High bearing resistance • Negligible settlement • Similar performance to existing piles 	<ul style="list-style-type: none"> • More costly than shallow footings 	<ul style="list-style-type: none"> • More expensive than shallow foundations • Estimated cost \$70,000 assuming twelve 13 metre long piles per pier and \$40,000 assuming four 15 metre long piles per abutment. 	<ul style="list-style-type: none"> • Possible pile tip damage if piles are not adequately protected while driving through very dense/hard soils • Embankment widenings should be constructed as far in advance of pile installation as possible to reduce potential for downdrag loads.

COMPARISON OF FOUNDATION ALTERNATIVES

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
End bearing steel H-pile foundations driven to refusal	<ul style="list-style-type: none"> • Feasible 	<ul style="list-style-type: none"> • High bearing resistance • Negligible settlement • Less potential vibration related impact compared to steel tube piles 	<ul style="list-style-type: none"> • Differential performance relative to existing tube piles 	<ul style="list-style-type: none"> • Estimated cost \$91,000 assuming twelve 21 metre long piles per pier and \$63,000 assuming four 19 metre long piles per abutment • More expensive than shallow foundations; cost competitive with tube piles 	<ul style="list-style-type: none"> • Possible pile tip damage if piles are not adequately protected while driving through very dense/hard soils. • Embankment widenings should be constructed as far in advance of pile installation as possible to reduce potential for downdrag loads.

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

Prepared By: DUP
 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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split spoon sampler for a distance of 300 mm (12 in.)

Consistency

	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

Dynamic Cone Penetration Resistance; N_d :

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

RECORD OF BOREHOLE No 301

1 OF 2

METRIC

PROJECT 08-1132-084-1

W.P. 131-98-00

LOCATION N 4809767.7 ; E 223606.9

ORIGINATED BY MA

DIST HWY 7/8

BOREHOLE TYPE POWER AUGER / ROTARY DRILLING / TRICONE

COMPILED BY LMK

DATUM GEODETIC

DATE October 8, 2008 - October 9, 2008

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
333.67	GROUND SURFACE							20 40 60 80 100						GR SA SI CL
0.00	FILL, sand and gravel, some silt Brown						Concrete							
333.21							Bentonite							
0.46	FILL, sandy silt, some gravel, some clay Dense to very dense Brown		1	SS	41									
331.81			2	SS	56/ 225mm									
1.86	ASPHALT													
331.98	FILL, sand and gravel Brown													
2.29	FILL, sandy silt, trace gravel, trace topsoil with clayey silt layers Very loose to compact Brown		3	SS	17		Backfill							
330.25			4	SS	3									
3.42	TOPSOIL, silty Loose Black						Standpipe							0 31 51 18
329.65	CLAYEY SILT, trace sand Firm Brown and grey mottled		5	SS	8									
4.02														
329.25	CLAYEY SILT TILL, trace to some sand, trace gravel Very stiff to hard Brown to grey at about elev. 327.0m		6	SS	19									
4.42														
			7	SS	27									1 15 46 38
			8	SS	35									
			9	SS	54									
			10	SS	22									
324.23							Bentonite							
9.44	CLAYEY SILT, trace sand with silt partings Stiff to very stiff Grey		11	SS	20									0 8 49 43
			12	SS	10									
321.17														
12.50	SAND AND GRAVEL Very dense Grey													
320.38			13	SS	84									
13.29	SANDY SILT, with clayey silt partings Very dense Grey													
319.80														
13.87	SAND, fine, some silt Very dense Grey		14	SS	123		Backfill							1 79 14 6

Continued Next Page

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

RECORD OF BOREHOLE No 302

1 OF 2

METRIC

PROJECT 08-1132-084-1
W.P. 131-98-00 LOCATION N 4809733.9 ; E 223622.9 ORIGINATED BY MA
DIST HWY 7/8 BOREHOLE TYPE POWER AUGER / ROTARY DRILLING / TRICONE COMPILED BY LMK
DATUM GEODETIC DATE October 27, 2008 - October 28, 2008 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE					w _p w w _L				
333.52	GROUND SURFACE						20	40	60	80	100	10	20	30	GR SA SI CL		
0.08	TOPSOIL, silty Brown FILL, silty sand and gravel, asphalt Compact Brown						333								1 65 27 7		
			1	SS	18												
332.15																	
1.37	FILL, sand, fine to medium, trace to some silt, trace clay, trace topsoil Loose to compact Brown		2	SS	25		332										
329.86																	
3.66	FILL, sandy silt, some clay, trace gravel Loose Brown																
329.10																	
4.42	CLAYEY SILT TILL, trace sand, trace gravel, with silty sand layers Very stiff Brown to grey																
			6	SS	26		329										
327.58																	
5.94	SILTY FINE SAND Dense Grey																
			8	SS	31	327											
326.51																	
7.01	SILT, some sand, some clay with clayey silt partings Compact Grey																
			9	SS	15	326											
324.83																	
8.69	SILTY FINE SAND, trace gravel Compact Grey																
			10	SS	25	325											
324.04																	
9.48	SILTY CLAY, trace sand with silt partings Very stiff Grey																
323.31																	
10.21	SILT, trace clay, with sand and clayey silt layers Compact Grey																
			11	SS	21	323											
321.94																	
11.58	CLAYEY SILT, trace sand, trace gravel, with silt partings Very stiff to hard Grey																
			12	SS	26	321											
319.62																	
13.90	SANDY SILT Very dense Grey																
			13	SS	53	320											
318.89																	
14.63																	

Continued Next Page

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

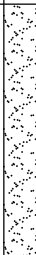

LDN_MTO_01 08-1132-084-1.GPJ LDN_MTO.GDT 01/09/10

RECORD OF BOREHOLE No 302

2 OF 2

METRIC

PROJECT 08-1132-084-1
W.P. 131-98-00 LOCATION N 4809733.9 ; E 223622.9 ORIGINATED BY MA
DIST HWY 7/8 BOREHOLE TYPE POWER AUGER / ROTARY DRILLING / TRICONE COMPILED BY LMK
DATUM GEODETIC DATE October 27, 2008 - October 28, 2008 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)					
								○ UNCONFINED + FIELD VANE	w _p w w _L							
						● QUICK TRIAXIAL × LAB VANE										
						20 40 60 80 100								GR SA SI CL		
	SAND, fine to medium Dense Grey						318								0 90 7 3	
			14	SS	49											
			15	SS	40											
315.99							317									
	</															

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

RECORD OF BOREHOLE No 303

1 OF 2

METRIC

PROJECT 08-1132-084-1

W.P. 131-98-00

LOCATION N 4809727.5 ; E 223602.4

ORIGINATED BY JB

DIST HWY 7/8

BOREHOLE TYPE POWER AUGER / ROTARY DRILLING / TRICONE

COMPILED BY LMK

DATUM GEODETIC

DATE November 3, 2008

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)			
								○ UNCONFINED		+ FIELD VANE									
								● QUICK TRIAXIAL		× LAB VANE									
333.17	GROUND SURFACE						20	40	60	80	100								
0.00	TOPSOIL, silty																		
0.15	Black FILL, silty fine sand, trace gravel Dense Brown																		
331.65																			
1.52	CLAYEY SILT, trace sand, trace gravel Very stiff Brown		2	SS	16														
				3	SS	24													
330.12																			
3.05	SILTY FINE SAND, with clayey silt layers Compact to dense Brown		4	SS	30														
				5	SS	17													
				6	SS	47													
				7	SS	40													
			8	SS	19														
326.16																			
7.01	SAND, fine to medium, some silt Very dense Grey																		
				9	SS	58													
324.48																			
8.69	SILTY FINE SAND Very dense Grey																		
				10	SS	102													
			11	SS	110														
321.59																			
11.58	SILTY CLAY, trace sand Very stiff to hard Grey																		
				12	SS	30													
			13	SS	68														
318.69																			
14.48	SANDY SILT, trace clay Very dense Grey																		

Continued Next Page

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

LDN_MTO_01 08-1132-084-1.GPJ LDN_MTO.GDT 01/09/10

PROJECT <u>08-1132-084-1</u>		RECORD OF BOREHOLE No 303		2 OF 2		METRIC	
W.P. <u>131-98-00</u>		LOCATION <u>N 4809727.5 ; E 223602.4</u>		ORIGINATED BY <u>JB</u>			
DIST <u> </u> HWY <u>7/8</u>		BOREHOLE TYPE <u>POWER AUGER / ROTARY DRILLING / TRICONE</u>		COMPILED BY <u>LMK</u>			
DATUM <u>GEODETIC</u>		DATE <u>November 3, 2008</u>		CHECKED BY <u> </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)								
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					W _p	W	W _L		GR	SA	SI	CL	
							20	40	60	80	100										
317.02	SANDY SILT, trace clay Very dense Grey		14	SS	83	▽	318														
16.15	SAND, fine to medium, trace gravel, trace silt Very dense Grey		15	SS	110		317														
315.49							316														
17.68	SILTY CLAY, trace sand with silt partings Hard Grey		16	SS	106		315														
							314														
312.90			17	SS	111		313														
20.27	END OF BOREHOLE Groundwater encountered at about elev. 317.9 during drilling on November 3, 2009.																				

RECORD OF BOREHOLE No 304

1 OF 1

METRIC

PROJECT 08-1132-084-1
W.P. 131-98-00 LOCATION N 4809762.2 ; E 223585.9 ORIGINATED BY JB
DIST HWY 7/8 BOREHOLE TYPE POWER AUGER / ROTARY DRILLING / TRICONE COMPILED BY LMK
DATUM GEODETIC DATE November 4, 2008 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _P	W	W _L		GR SA SI CL					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)										
333.46	GROUND SURFACE							20	40	60	80	100							
0.00	TOPSOIL							20	40	60	80	100							
0.15	FILL, clayey silt, trace to some sand and gravel Very stiff Brown (Hydrocarbon odour)						333							18	29	35	18		
			1	SS	27														
			2	SS	20		332												
			3	SS	15		331												
			4	SS	17		330												
329.80	CLAYEY SILT, trace sand, trace gravel Stiff to hard Brown		5	SS	32		329												
3.66			6	SS	39		328							1	18	43	38		
			7	SS	35		327												
			8	SS	46		326												
			9	SS	42		325												
			10	SS	9		324												
323.55	SANDY SILT, trace clay Very dense Grey		11	SS	102		323												
9.91							322												
321.27	SAND, fine to medium, trace silt Very dense Grey		12	SS	103		321									1	90	3	6
12.19							320												
319.29	END OF BOREHOLE		13	SS	101														
14.17	Borehole dry during drilling on November 4, 2009.																		

LDN_MTO_01_08-1132-084-1.GPJ LDN_MTO_GDT_01/09/10

RECORD OF BOREHOLE No 305

1 OF 1

METRIC

PROJECT 08-1132-084-1

W.P. 131-98-00

LOCATION N 4809723.6 ; E 223574.7

ORIGINATED BY JB

DIST HWY 7/8

BOREHOLE TYPE POWER AUGER / HOLLOW STEM

COMPILED BY LMK

DATUM GEODETIC

DATE November 17, 2008

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				W _p W W _L							
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)										
339.23	PAVEMENT SURFACE						20	40	60	80	100					GR	SA	SI	CL
0.00	ASPHALT																		
0.15	FILL, sand and gravel, some silt Compact Brown																		
337.86			1	SS	20														
1.37	FILL, clayey silt with sandy silt layers, trace gravel Very stiff Brown		2	SS	23														
			3	SS	22														
336.33																			
2.90	SANDY SILT, with clayey silt layers Dense Brown		4	SS	34														
			5	SS	47														
334.81																			
4.42	CLAYEY SILT, some sand Very stiff Grey and brown		6	SS	17														
333.59																			
5.64	SANDY SILT, trace clay with clayey silt layers Dense Grey		7	SS	45														
332.22																			
7.01	CLAYEY SILT TILL, trace sand, trace gravel Very stiff Grey		8	SS	28														
331.15																			
8.08	END OF BOREHOLE																		
	Borehole dry during drilling on November 17, 2008.																		

RECORD OF BOREHOLE No 306

1 OF 2

METRIC

PROJECT 08-1132-084-1

W.P. 131-98-00

LOCATION N 4809725.5 ; E 223583.0

ORIGINATED BY JB

DIST HWY 7/8

BOREHOLE TYPE POWER AUGER / ROTARY DRILLING / TRICONE

COMPILED BY LMK

DATUM GEODETIC

DATE November 17, 2008 - November 18, 2008

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE									
339.17	PAVEMENT SURFACE							20	40	60	80	100						
0.00	ASPHALT																	
0.15	FILL, sand and gravel, trace silt Loose Brown						339											
337.80			1	SS	8													
337.80	FILL, clayey silt, some sand, trace gravel Stiff Brown						338											
1.37			2	SS	10													
336.27			3	SS	12		337											
336.27																		
2.90	SANDY SILT, trace clay with clayey silt layers Compact Brown		4	SS	22		336											
			5	SS	20		335						○		0	33 48 19		
			6	SS	28		334											
333.23																		
5.94	SILTY SAND AND GRAVEL Very dense Brown		8	SS	110/ 250mm		333											
332.16	SANDY SILT, trace clay Compact to very dense Brown						332											
7.01			9	SS	13		331											
			10	SS	56		330											
329.11																		
10.06	SAND, fine to coarse, some silt Very dense Grey						329											
			11	SS	65		328						○		2	84 10 4		
327.59																		
11.58	SILTY CLAY, with sandy silt layers Hard Grey		12	SS	86		327											
			13	SS	37		326											
324.54							325						○					
14.63																		

Continued Next Page

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

LDN_MTO_01_08-1132-084-1.GPJ LDN_MTO.GDT 01/09/10

PROJECT <u>08-1132-084-1</u>		RECORD OF BOREHOLE No 306		2 OF 2		METRIC	
W.P. <u>131-98-00</u>		LOCATION <u>N 4809725.5 ; E 223583.0</u>		ORIGINATED BY <u>JB</u>			
DIST <u> </u> HWY <u>7/8</u>		BOREHOLE TYPE <u>POWER AUGER / ROTARY DRILLING / TRICONE</u>		COMPILED BY <u>LMK</u>			
DATUM <u>GEODETIC</u>		DATE <u>November 17, 2008 - November 18, 2008</u>		CHECKED BY <u> </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL		
								20	40	60	80	100	W _p	W	W _L							
	CLAYEY SILT, trace sand Hard Grey		14	SS	100/ 150mm													0	5	70	25	
				15	SS	117																
321.49																						
17.68	SILTY CLAY, trace sand Hard Grey																					
320.42			16	SS	103																	
18.75	END OF BOREHOLE																					
	Groundwater level encountered at about elev. 330.0m during drilling on November 18, 2009.																					

LDN_MTO_01 08-1132-084-1.GPJ LDN_MTO.GDT 01/09/10

RECORD OF BOREHOLE No 307

1 OF 2

METRIC

PROJECT 08-1132-084-1

W.P. 131-98-00

LOCATION N 4809744.1 ; E 223643.8

ORIGINATED BY JB

DIST HWY 7/8

BOREHOLE TYPE POWER AUGER / HOLLOW STEM

COMPILED BY LMK

DATUM GEODETIC

DATE November 19, 2008

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								20	40	60	80	100			W _P	W	W _L
338.45	PAVEMENT SURFACE																
0.00	ASPHALT																
0.15	FILL, sand and gravel																
337.84	Brown																
0.61	FILL, clayey silt, trace sand, trace gravel, with silt and sand layers		1	SS	15												
	Stiff to hard		2	SS	13												
	Brown		3	SS	23												
			4	SS	12												
			5	SS	10												
			6	SS	20												
			7	SS	18												
			8	SS	26												
			9	SS	24												
			10	SS	33												
330.22	SANDY SILT		11	SS	45												
8.23	Dense		12	SS	51												
329.46	SANDY SILT, with peat layers, trace gravel																
8.99	Very dense																
	Grey and black																
328.39	SILTY FINE SAND, trace silt, trace clay		13	SS	38												
10.06	Dense		14	SS	46												
	Grey																
325.34	CLAYEY SILT, trace sand, with silt partings		15	SS	23												
13.11	Very stiff																
	Grey																
323.82																	
14.63																	

Continued Next Page

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

LDN_MTO_01_08-1132-084-1.GPJ LDN_MTO.GDT 01/09/10

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

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

PROJECT <u>08-1132-084-1</u>		RECORD OF BOREHOLE No 308		2 OF 2		METRIC	
W.P. <u>131-98-00</u>		LOCATION <u>N 4809769.0 ; E 223627.1</u>		ORIGINATED BY <u>JB</u>			
DIST <u> </u> HWY <u>7/8</u>		BOREHOLE TYPE <u>POWER AUGER / ROTARY DRILLING / TRICONE</u>		COMPILED BY <u>LMK</u>			
DATUM <u>GEODETIC</u>		DATE <u>November 24, 2008</u>		CHECKED BY <u> </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	w _p	w	w _L		

RECORD OF BOREHOLE No 309

1 OF 2

METRIC

PROJECT 08-1132-084-1
W.P. 131-98-00 LOCATION N 4809771.7 ; E 223638.0 ORIGINATED BY MA
DIST HWY 7/8 BOREHOLE TYPE POWER AUGER / HOLLOW STEM COMPILED BY DMB
DATUM GEODETIC DATE June 2, 2009 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED		+ FIELD VANE							
								● QUICK TRIAXIAL		× LAB VANE							
338.36	GROUND SURFACE					20	40	60	80	100							
0.00	ASPHALT																
0.18	FILL, sand and gravel, crushed																
0.40	FILL, sand and gravel, with cobbles																
337.63	Brown																
0.73	FILL, clayey silt, trace to some sand, trace gravel, with silt layers Firm to very stiff Brown		1	SS	13												
			2	SS	15												
			3	SS	15												
			4	SS	6												
			5	SS	13												
333.79																	
4.57	FILL, sand and gravel, trace silt, asphalt fragments Compact Grey		6	SS	26												
333.18																	
5.18	FILL, clayey silt, trace sand, trace gravel, with silt layers Very stiff Brown		7	SS	21												
			8	SS	19												
331.65																	
6.71	SILTY FINE SAND, trace gravel, trace clay Very loose to dense Brown		9	SS	28												
			10	SS	41												
			11	SS	14												
			12	SS	2												
328.61																	
9.75	ORGANIC SILT Soft to very stiff Grey		13	SS	2												
327.33			14	SS	19												
11.03	SANDY SILT, trace clay Compact Grey																
326.78																	
11.58	CLAYEY SILT, trace to some sand, trace gravel Stiff to very stiff		15	SS	19												
			16	SS	14												

Continued Next Page

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

LDN_MTO_01 08-1132-084-1.GPJ LDN_MTO.GDT 01/09/10

PROJECT <u>08-1132-084-1</u>		RECORD OF BOREHOLE No 309		2 OF 2		METRIC	
W.P. <u>131-98-00</u>		LOCATION <u>N 4809771.7 ; E 223638.0</u>		ORIGINATED BY <u>MA</u>			
DIST <u> </u> HWY <u>7/8</u>		BOREHOLE TYPE <u>POWER AUGER / HOLLOW STEM</u>		COMPILED BY <u>DMB</u>			
DATUM <u>GEODETIC</u>		DATE <u>June 2, 2009</u>		CHECKED BY <u> </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
							20	40	60	80	100	10	20	30						
	CLAYEY SILT, trace to some sand, trace gravel Stiff to very stiff		17	SS	15	▽	323									1	2	47	50	
								322												
				18	SS		14	321												
320.68							320													
17.68	SANDY SILT, trace clay Compact Grey		19	SS	10															
				20	SS	7	319													
319.13	CLAYEY SILT, trace sand Firm Grey																			
19.23	END OF BOREHOLE																			
19.51	Groundwater encountered at about elev. 329.3m and elev. 320.7m during drilling on June 2, 2009.																			

LDN_MTO_01 08-1132-084-1.GPJ LDN_MTO.GDT 01/09/10

RECORD OF BOREHOLE No 310

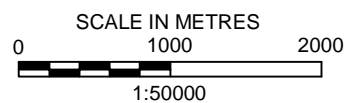
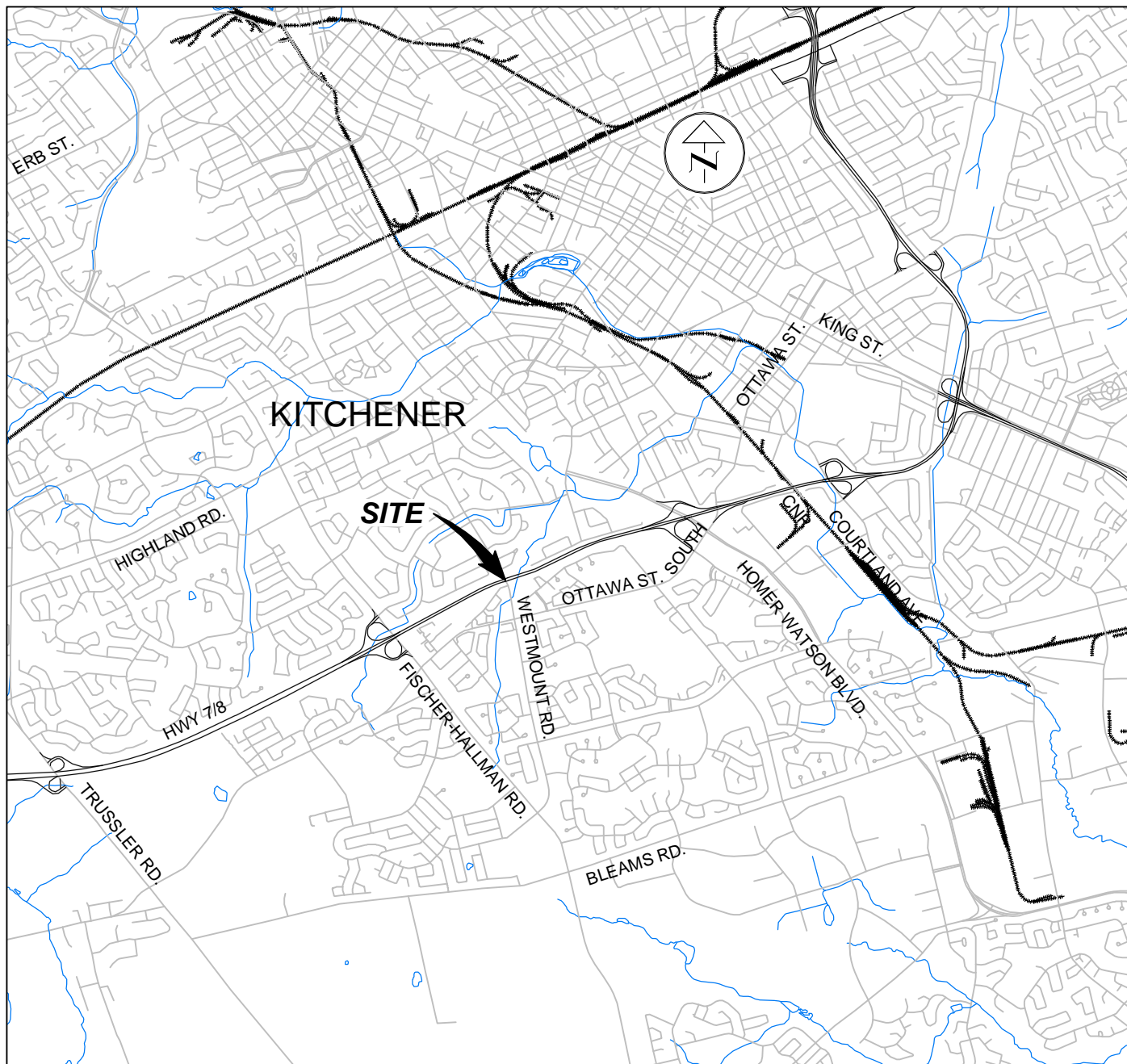
1 OF 1

METRIC

PROJECT 08-1132-084-1
W.P. 131-98-00 LOCATION N 4809748.2 ; E 223560.9 ORIGINATED BY MA
DIST HWY 7/8 BOREHOLE TYPE POWER AUGER / HOLLOW STEM COMPILED BY DMB
DATUM GEODETIC DATE June 3, 2009 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					W _P	W	W _L		WATER CONTENT (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
339.27	GROUND SURFACE					▽		20	40	60	80	100																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																</

LDN_MTO_01_08-1132-084-1.GPJ LDN_MTO.GDT 01/09/10



NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

PROJECT
WESTMOUNT ROAD OVERPASS (SITE No. 33-228)
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE

KEY PLAN



PROJECT No. 08-1132-084-1			FILE No. 0811320841-F03001		
			SCALE AS SHOWN		REV.
CADD	WDF/PH	Aug. 18/10	FIGURE 1		
CHECK					

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

CONT No.
WP No. 131-98-00

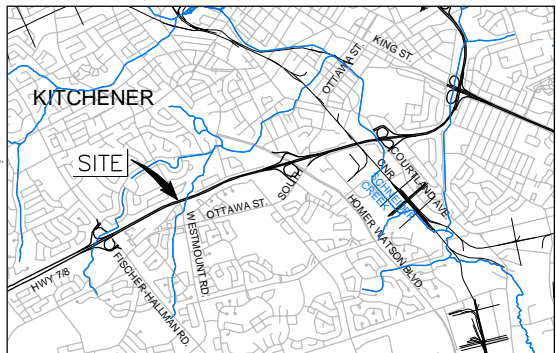


WESTMOUNT ROAD OVERPASS
WIDENING OF HIGHWAY 7/8
BOREHOLE LOCATIONS

SHEET



Golder Associates Ltd.
LONDON, ONTARIO, CANADA



KEY PLAN

SCALE IN KILOMETRES
0 1 2

LEGEND

- Borehole - Current Investigation
- Borehole and Cone Penetration Test (By Others)
(Geocres #40P08-031)

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
301	333.67	4 809 767.7	223 606.9
302	333.52	4 809 733.9	223 622.9
303	333.17	4 809 727.5	223 602.4
304	333.46	4 809 762.2	223 585.9
305	339.23	4 809 723.6	223 574.7
306	339.17	4 809 725.5	223 583.0
307	338.45	4 809 744.1	223 643.8
308	338.55	4 809 769.0	223 627.1
309	338.36	4 809 771.7	223 638.0
310	339.27	4 809 748.2	223 560.9
By Others (Geocres #40P08-031)			
4	332.99	4 809 752.7	223 572.1
8	330.04	4 809 737.8	223 633.6

NOTES

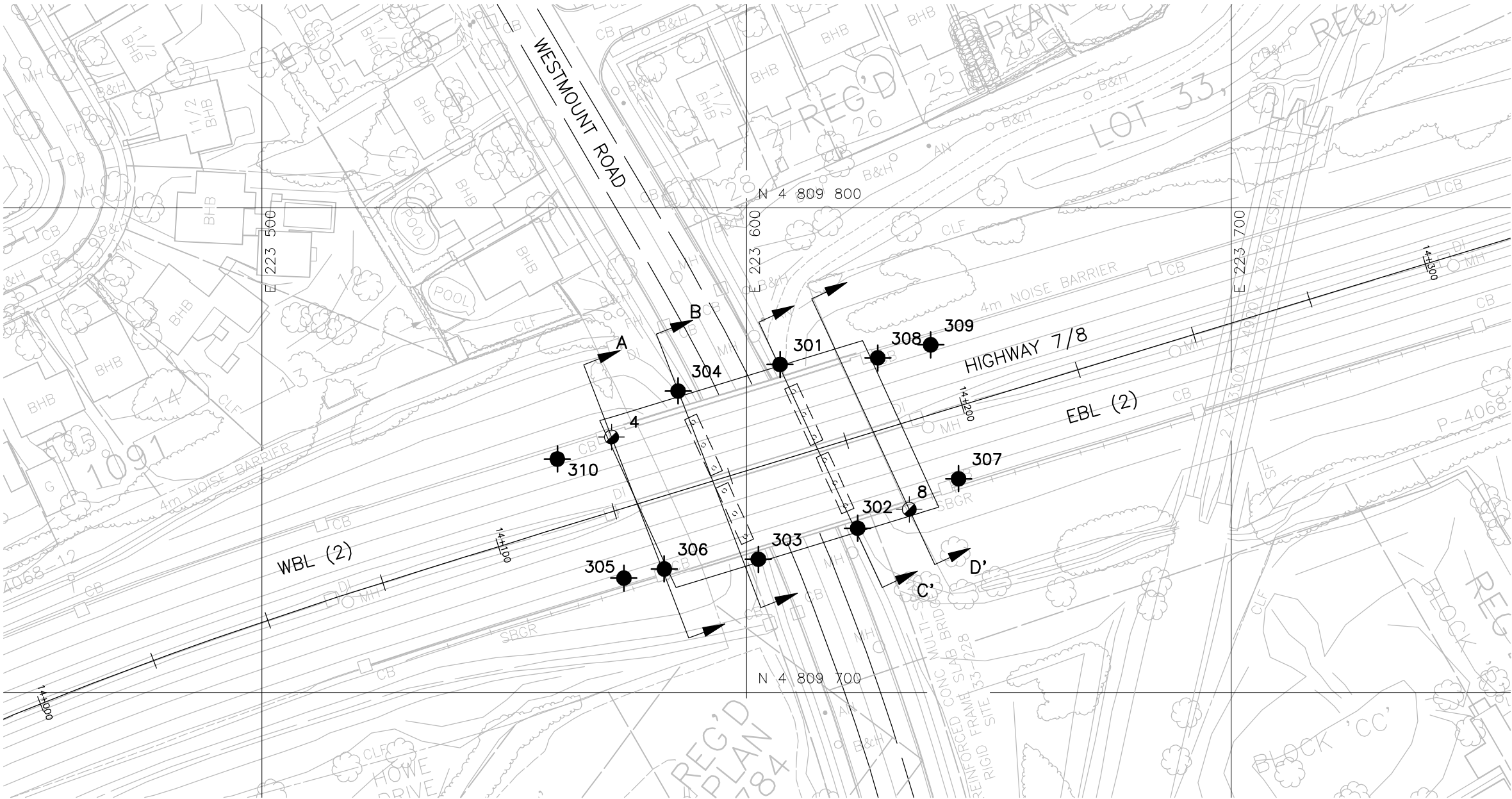
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

REFERENCE

Base plans provided in digital format by Dillon Consulting.

NO.	DATE	BY	REVISION
Geocres No. 40P7-55			
HWY.	7/8	PROJECT NO. 08-1132-084-1	DIST.
SUBM'D.	DUP	CHKD.	DATE: Aug. 18/10
DRAWN:	LMK	CHKD.	APPD.
			DWG. 1



PLAN

SCALE

10 0 10 m

LEGEND	
	Borehole - Current Investigation
	Borehole and Cone Penetration Test (By Others) (Geocres #40P08-031)
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
DRY	Borehole dry during drilling
	WL upon completion of drilling
	Measured WL

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
301	333.67	4 809 767.7	223 606.9
302	333.52	4 809 733.9	223 622.9
303	333.17	4 809 727.5	223 602.4
304	333.46	4 809 762.2	223 585.9
305	339.23	4 809 723.6	223 574.7
306	339.17	4 809 725.5	223 583.0
307	338.45	4 809 744.1	223 643.8
308	338.55	4 809 769.0	223 627.1
309	338.36	4 809 771.7	223 638.0
310	339.27	4 809 748.2	223 560.9
By Others (Geocres #40P08-031)			
4	332.99	4 809 752.7	223 572.1
8	330.04	4 809 737.8	223 633.6

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.

Stratigraphy has been simplified for clarity. Please refer to Record of Boreholes and sections for further detail.

REFERENCE

Base plans provided in digital format by Dillon Consulting.

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

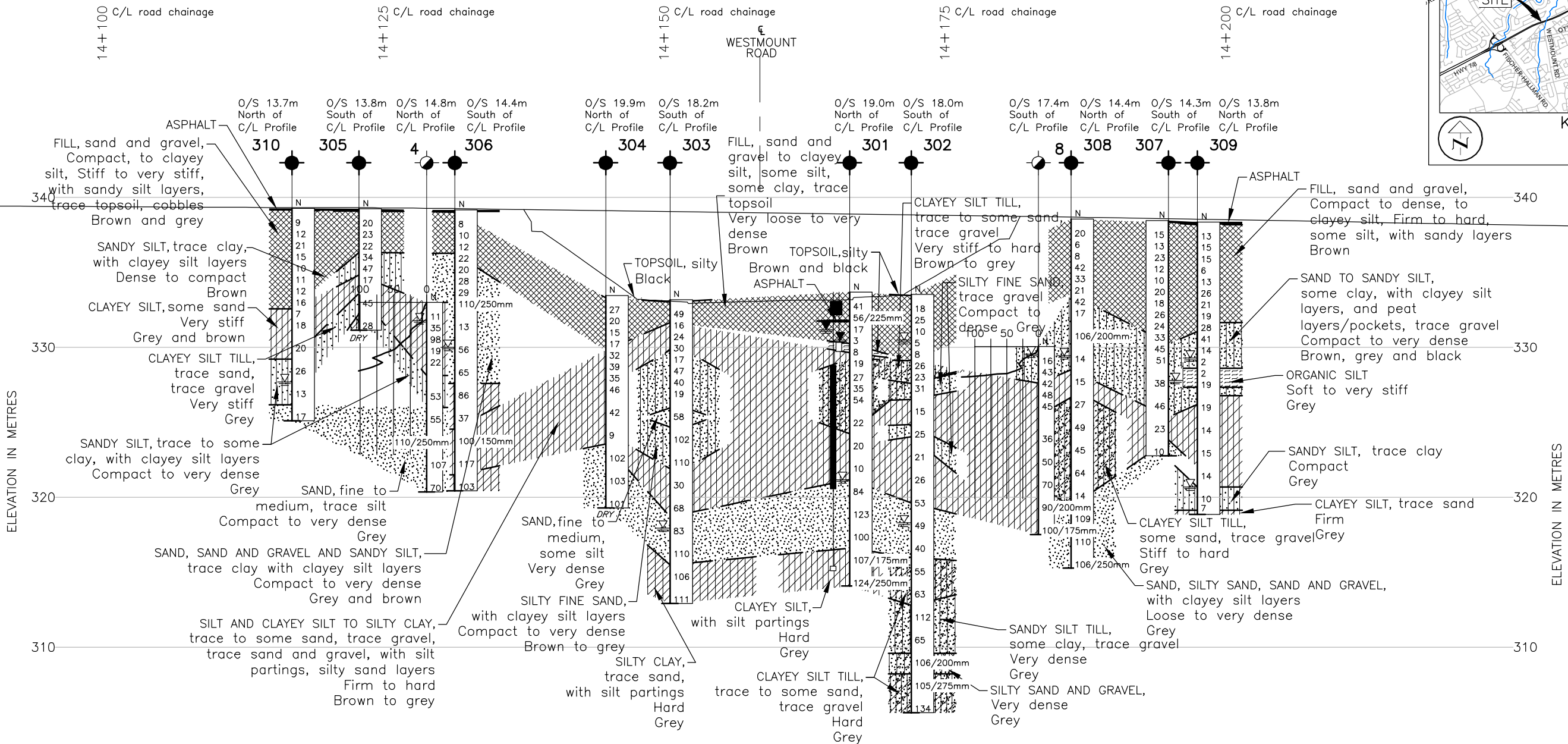
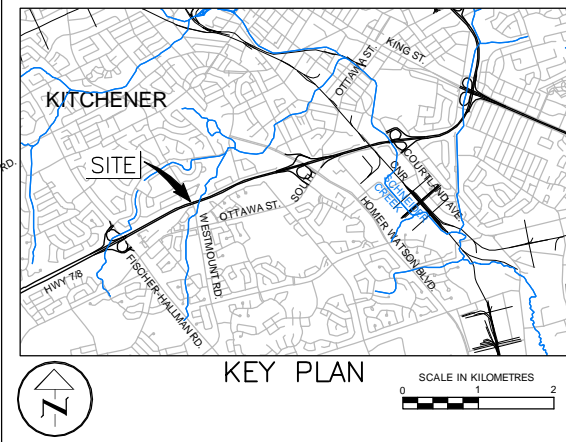
CONT No.
WP No. 131-98-00

WESTMOUNT ROAD OVERPASS
WIDENING OF HIGHWAY 7/8
SOIL STRATA

SHEET

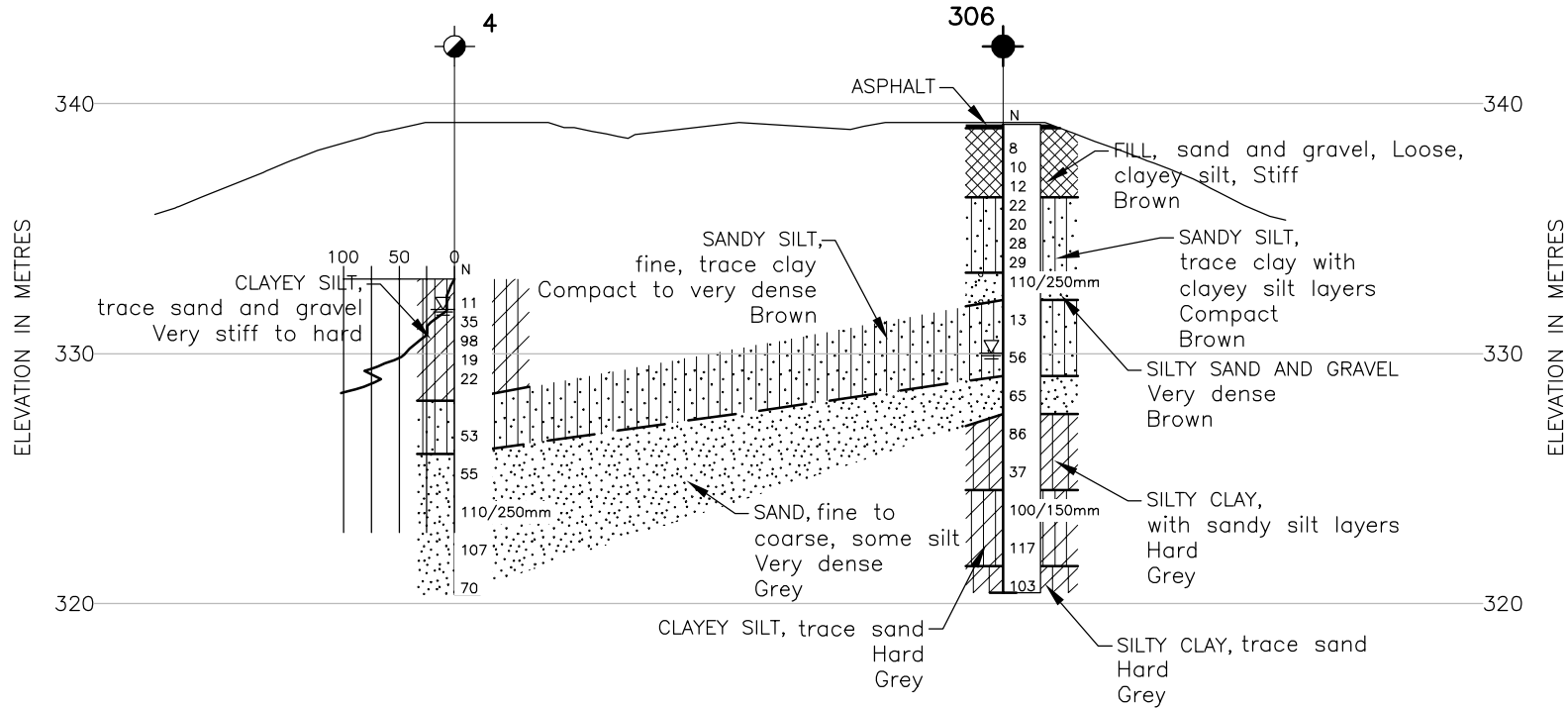


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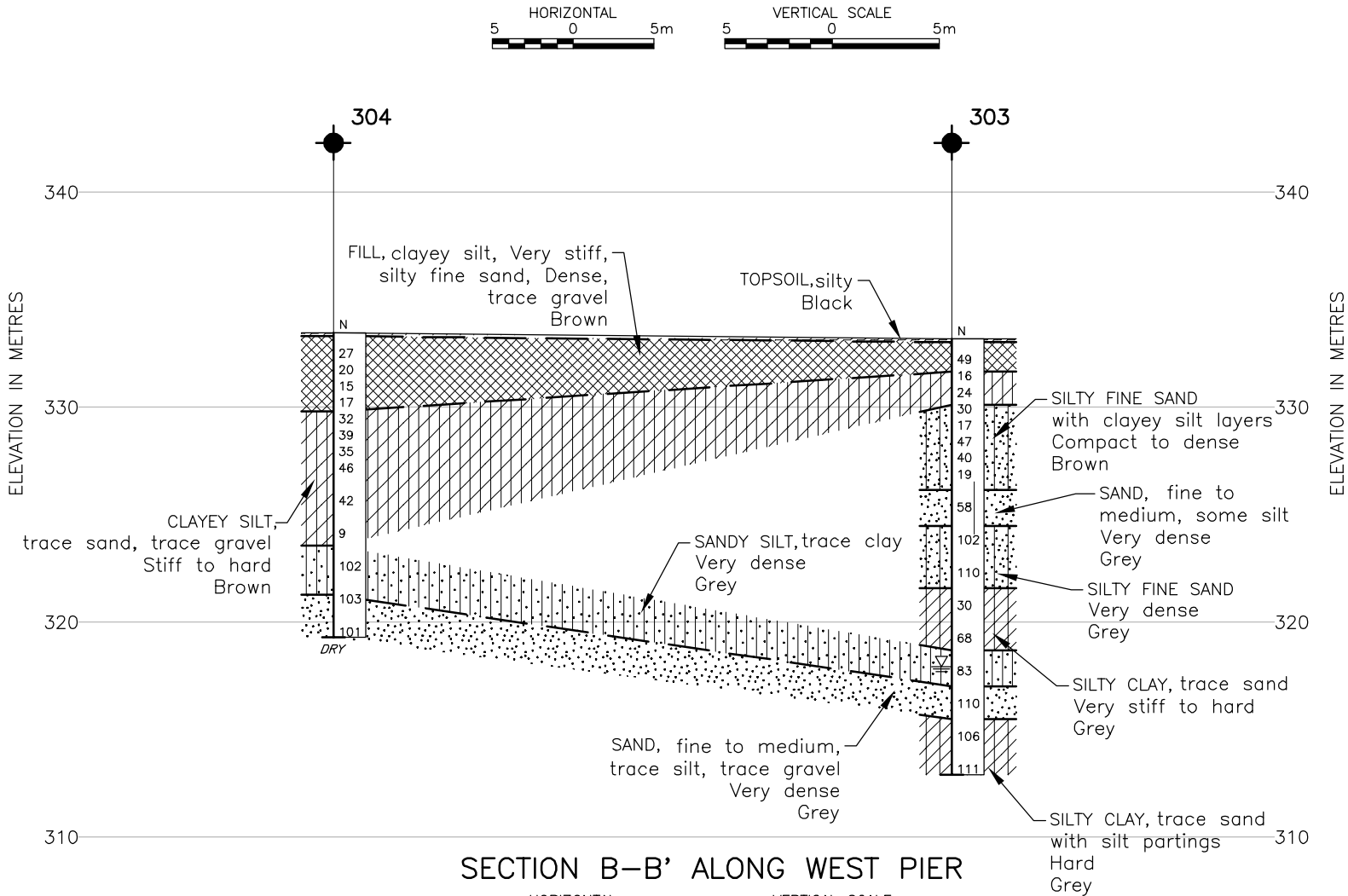


PROFILE ALONG C/L OF HIGHWAY 7/8

NO.	DATE	BY	REVISION
Geocres No. 40P7-55			
HWY.	7/8	PROJECT NO.	08-1132-084-1
SUBM'D.	DUP	CHKD.	DATE: Aug. 18/10
DRAWN:	LMK	CHKD.	APPD.
DIST.		SITE: 33-228	
DWG.		2	



SECTION A-A' ALONG WEST ABUTMENT



SECTION B-B' ALONG WEST PIER

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

CONT No.
WP No. 131-98-00

WESTMOUNT ROAD OVERPASS
WIDENING OF HIGHWAY 7/8
SOIL STRATA

SHEET



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LEGEND

- Borehole - Current Investigation
- Borehole (By Others) (Geocres #40P08-051)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- DRY Borehole dry during drilling
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
303	333.17	4 809 727.5	223 602.4
304	333.46	4 809 762.2	223 858.9
306	339.17	4 809 725.5	223 583.0
By Others (Geocres #40P08-031)			
4	332.99	4 809 752.7	223 572.1

NOTES

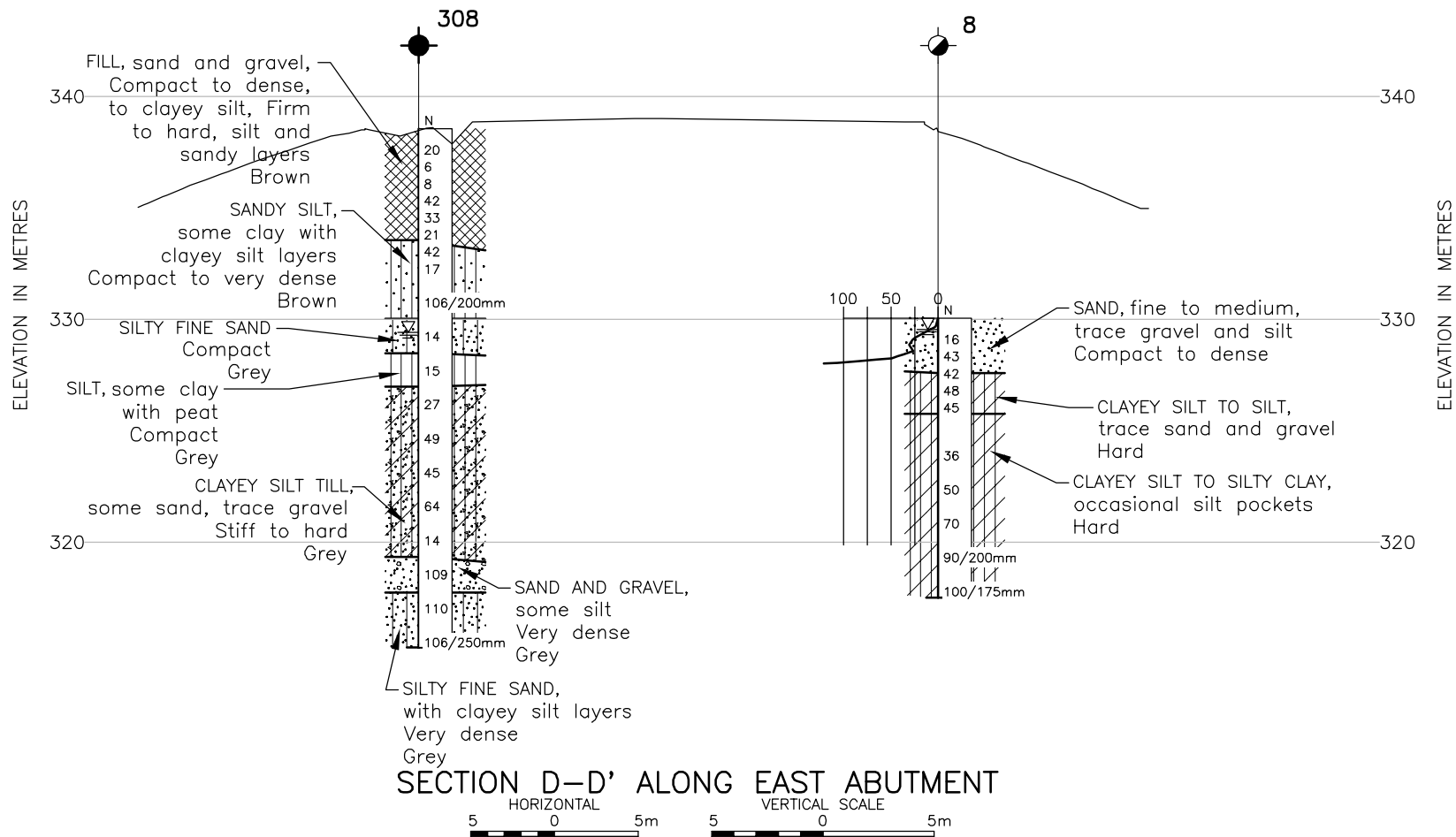
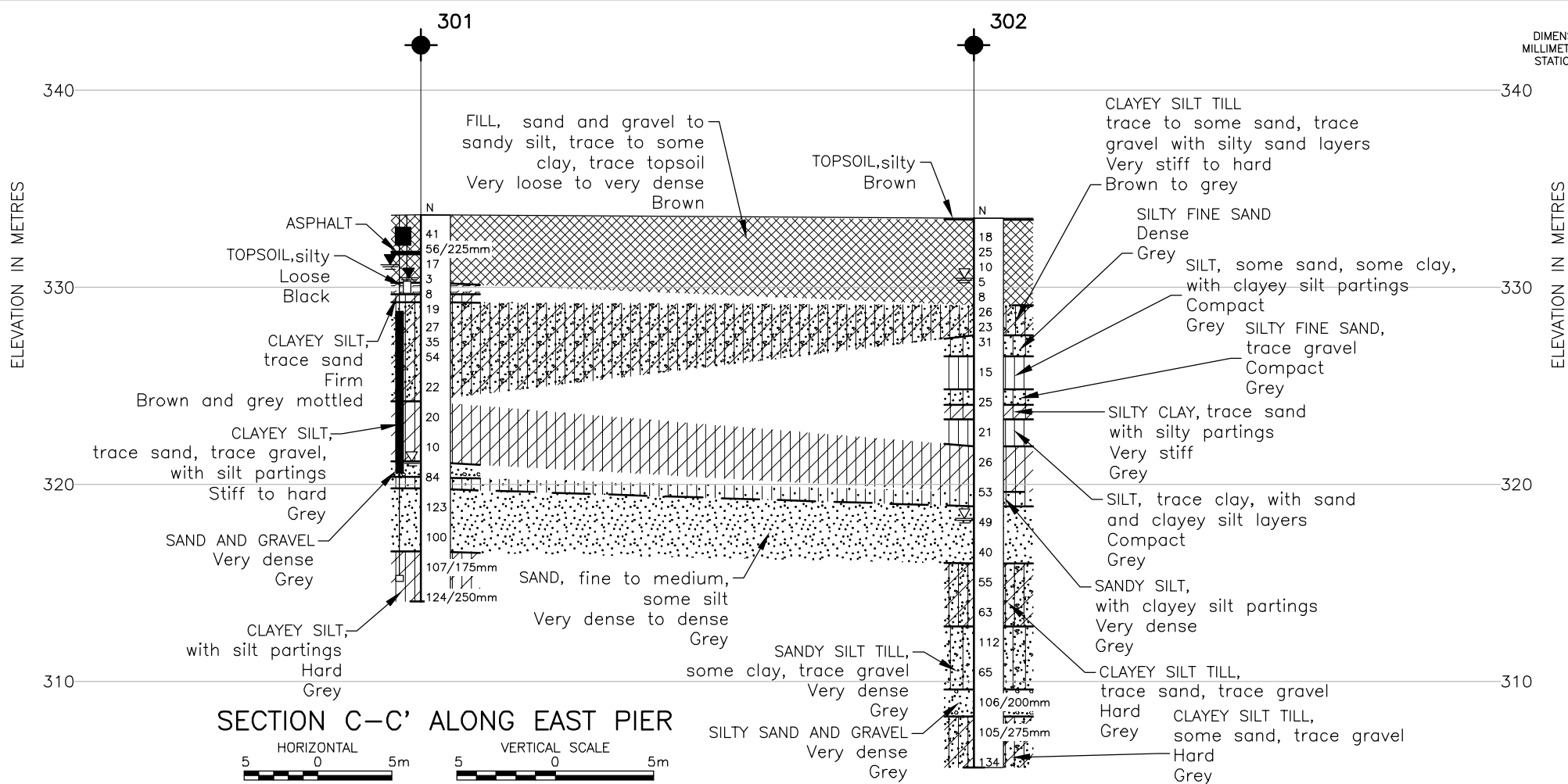
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

REFERENCE

Base plans provided in digital format by Dillon Consulting.

NO.	DATE	BY	REVISION
Geocres No. 40P7-55			
HWY.	7/8	PROJECT NO. 08-1132-084-1	DIST.
SUBM'D.	DUP	CHKD.	DATE: Aug. 18/10
DRAWN:	LMK	CHKD.	APPD.
			DWG. 3



CONT No.
WP No. 131-98-00

WESTMOUNT ROAD OVERPASS
WIDENING OF HIGHWAY 7/8
SOIL STRATA

SHEET



Golder Associates Ltd.
LONDON, ONTARIO, CANADA

LEGEND

- Borehole - Current Investigation
- Borehole (By Others) (Geocres #40P08-051)
- N** Standard Penetration Test Value
- 16** Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- DRY** Borehole dry during drilling
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
301	333.67	4 809 767.7	223 606.9
302	333.52	4 809 733.9	223 622.9
307	338.45	4 809 744.1	223 643.8
308	338.55	4 809 769.0	223 627.1

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

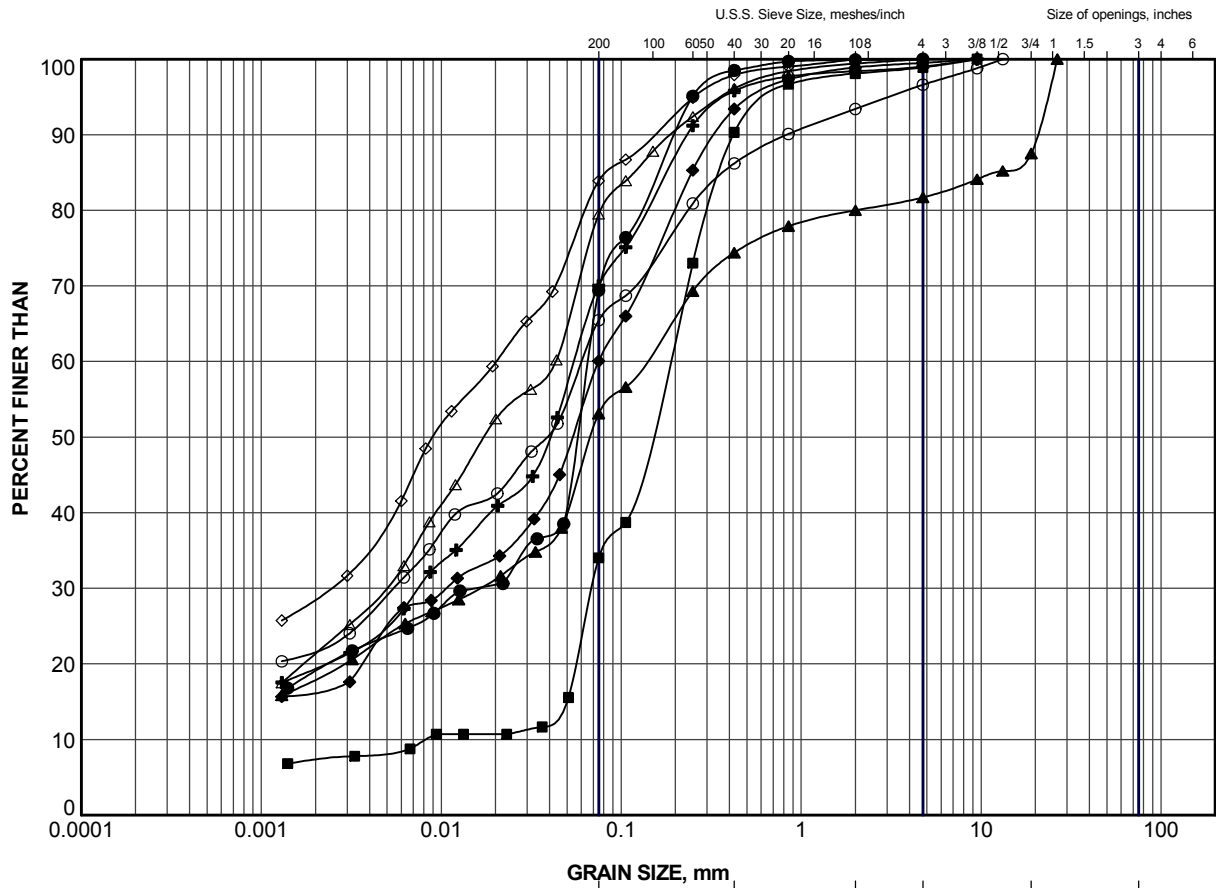
Base plans provided in digital format by Dillon Consulting.

NO.	DATE	BY	REVISION
Geocres No.	40P7-55		
HWY.	7/8	PROJECT NO.	08-1132-084-1
SUBM'D.	DUP	CHKD.	DATE: Aug. 19/10
DRAWN:	LMK	CHKD.	APPD.
			DWG. 4



APPENDIX A


Laboratory Test Data

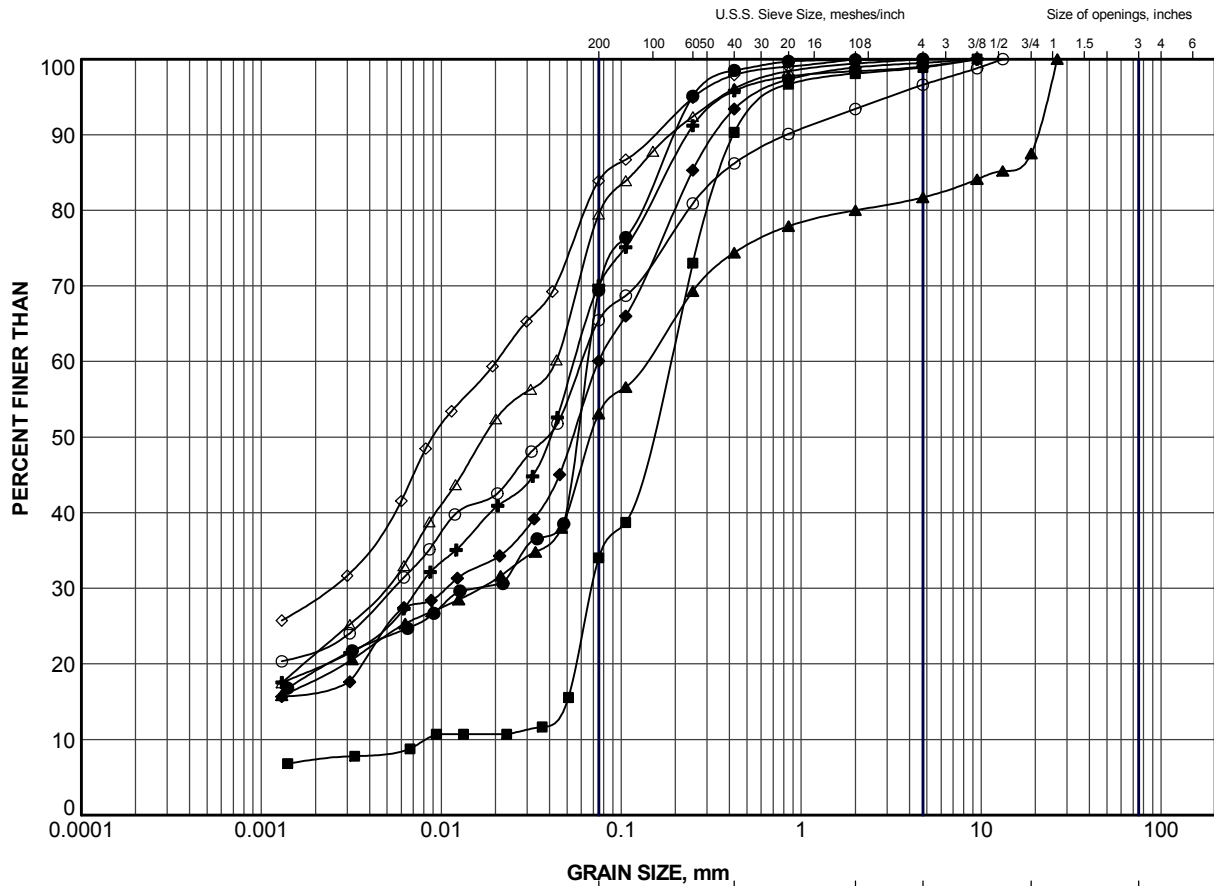


CLAY AND SILT	SAND SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	301	4	330.4
■	302	4	330.2
▲	304	1	332.5
+	305	2	337.5
◆	307	4	335.2
◇	307	9	331.4
○	308	3	336.0
△	309	5	334.3

PROJECT				WESTMOUNT ROAD OVERPASS (SITE No. 33-228) WIDENING OF HIGHWAY 7/8 GWP 131-98-00			
TITLE				GRAIN SIZE DISTRIBUTION FILL			
PROJECT No.		08-1132-084-1		FILE No.		0811320841-R030A1	
DRAWN		LMK		Aug 18/10		SCALE N/A REV.	
CHECK						FIGURE A-1	
 Golder Associates LONDON, ONTARIO							



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	301	4	330.4
■	302	4	330.2
▲	304	1	332.5
+	305	2	337.5
◆	307	4	335.2
◇	307	9	331.4
○	308	3	336.0
△	309	5	334.3

PROJECT WESTMOUNT ROAD OVERPASS (SITE No. 33-228)
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

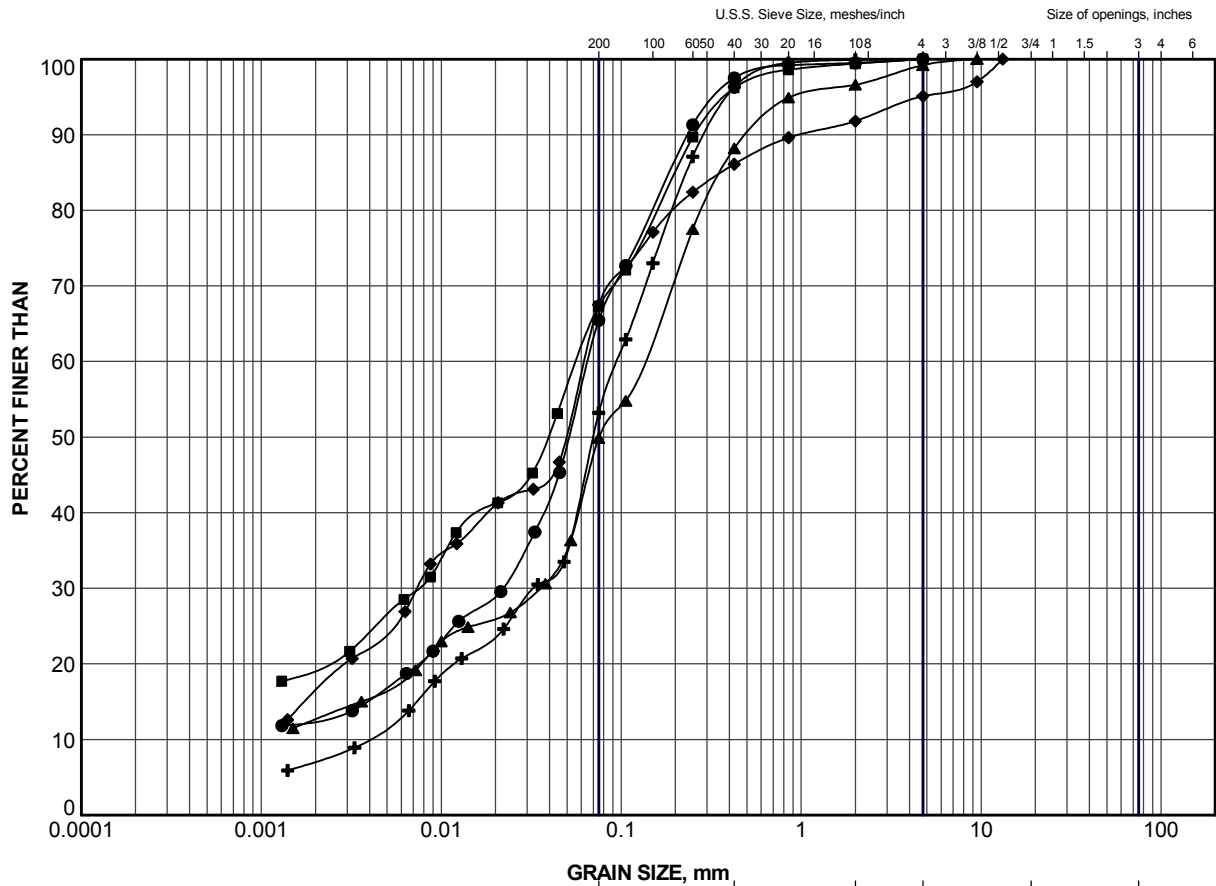
TITLE

GRAIN SIZE DISTRIBUTION FILL



PROJECT No.	08-1132-084-1	FILE No.	0811320841-R030A1
DRAWN	LMK	Aug 18/10	SCALE N/A REV.
CHECK			


FIGURE A-1

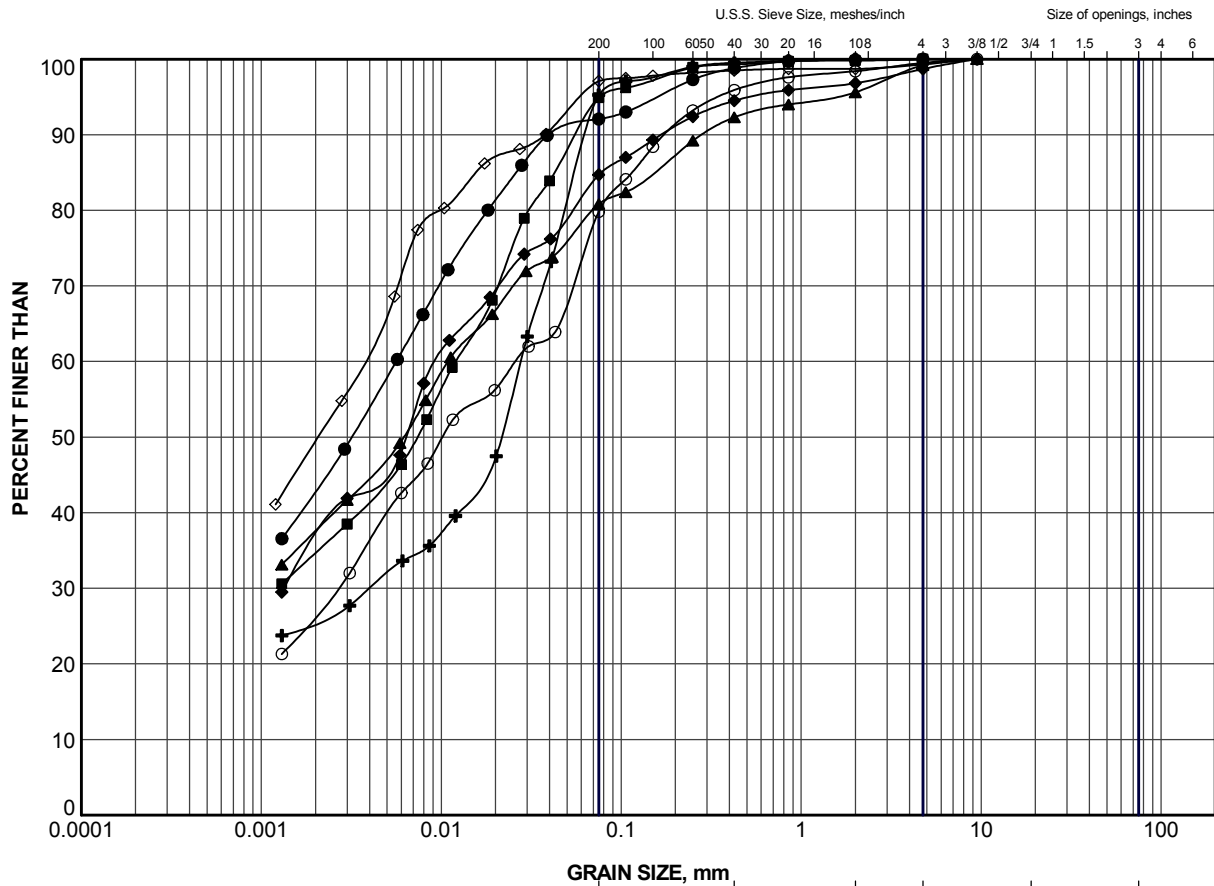


CLAY AND SILT	SAND SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	305	5	335.2
■	306	5	335.1
▲	308	9	330.7
+	309	19	319.8
◆	310	12	328.4

PROJECT					WESTMOUNT ROAD OVERPASS (SITE No. 33-228) WIDENING OF HIGHWAY 7/8 GWP 131-98-00				
TITLE					GRAIN SIZE DISTRIBUTION SANDY SILT				
PROJECT No.		08-1132-084-1		FILE No.		0811320841-R030A2			
DRAWN		LMK		Aug 19/10		SCALE		N/A	
CHECK						REV.			
 Golder Associates LONDON, ONTARIO					FIGURE A-2				



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	301	11	323.4
■	301	16	315.8
▲	304	7	327.9
+	306	14	323.7
◆	309	15	325.9
◇	309	17	322.9
○	310	10	331.4

PROJECT WESTMOUNT ROAD OVERPASS (SITE No. 33-228)
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

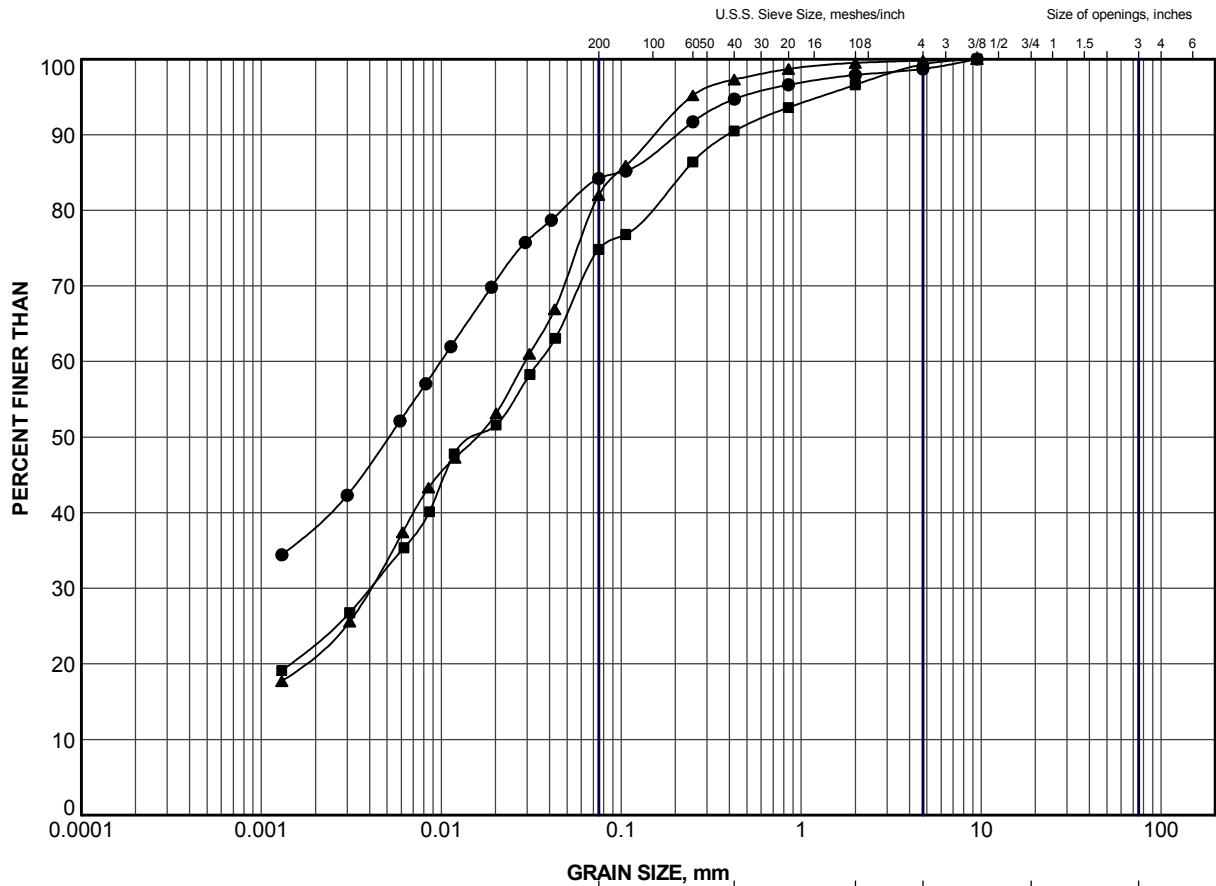
TITLE

GRAIN SIZE DISTRIBUTION CLAYEY SILT



PROJECT No.	08-1132-084-1	FILE No.	0811320841-R030A3
DRAWN	LMK	Aug 18/10	SCALE N/A REV.
CHECK			


FIGURE A-3

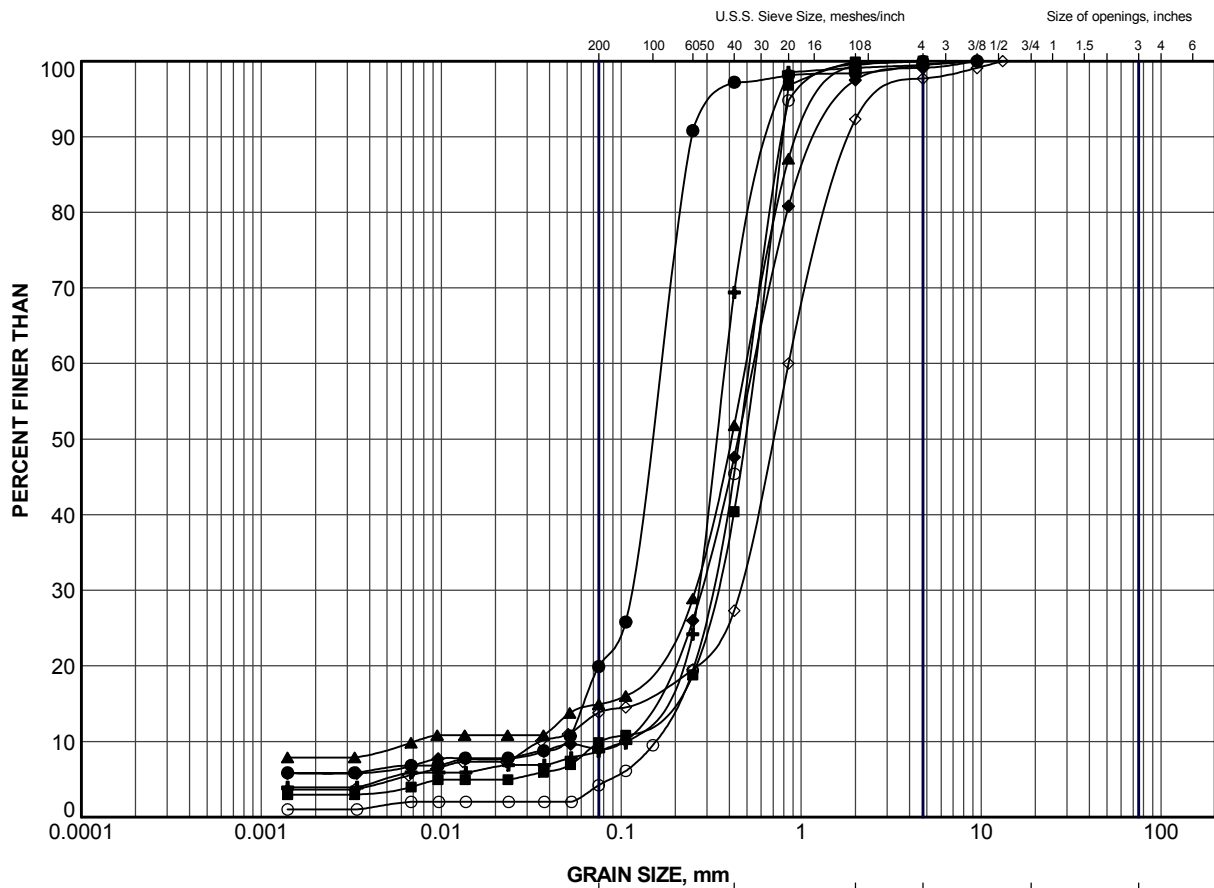


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	301	7	328.0
■	302	21	307.5
▲	308	12	326.1


PROJECT				WESTMOUNT ROAD OVERPASS (SITE No. 33-228) WIDENING OF HIGHWAY 7/8 GWP 131-98-00			
TITLE				GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL			
PROJECT No.		08-1132-084-1		FILE No.		0811320841-R030A4	
DRAWN		LMK		SCALE		N/A	
CHECK				REV.			
		Aug 19/10					
 Golder Associates LONDON, ONTARIO				FIGURE A-4			

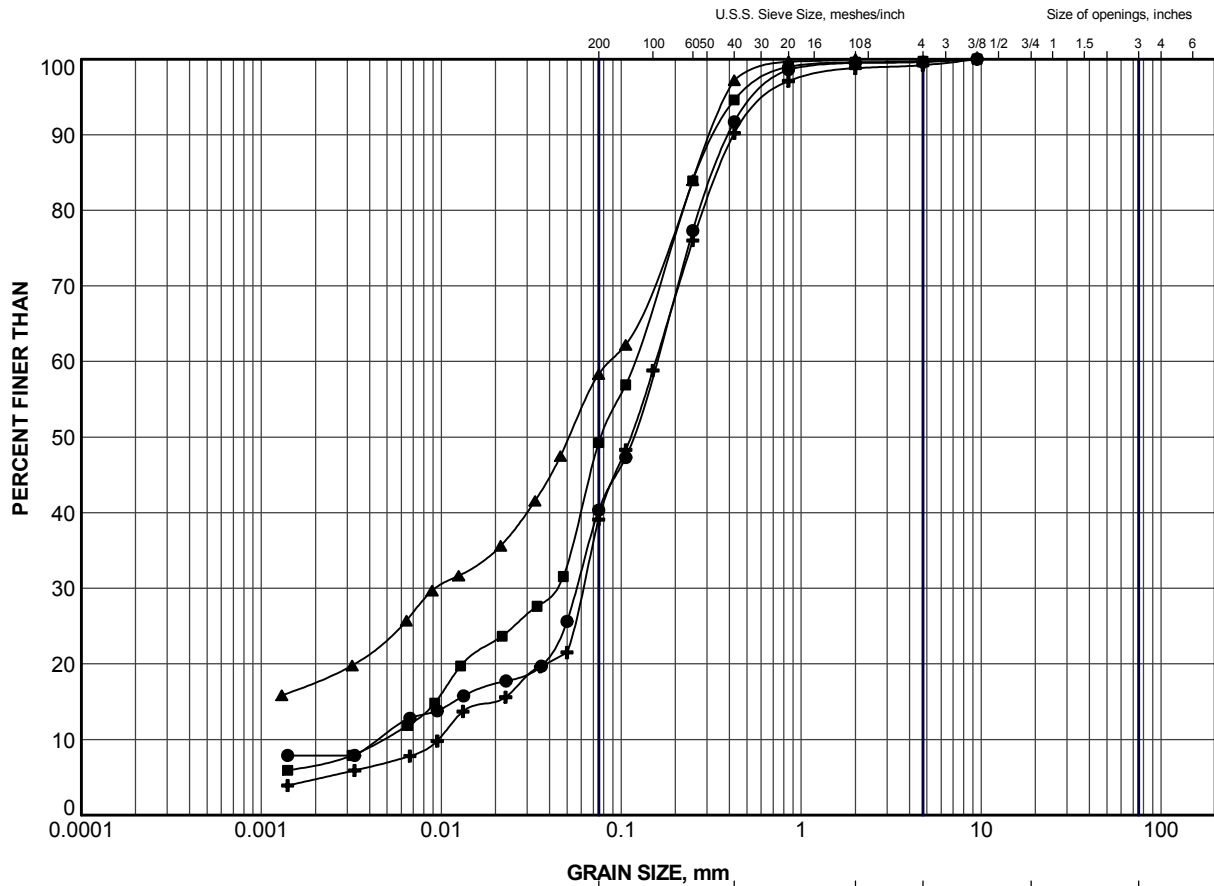


CLAY AND SILT		SAND SIZE, mm			GRAVEL SIZE, mm		Cobble Size
		fine	medium	coarse	fine	coarse	
		SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	301	14	318.8
■	302	14	318.1
▲	303	9	325.3
+	303	15	316.2
◆	304	12	321.0
◇	306	11	328.3
○	310	14	325.3


PROJECT				WESTMOUNT ROAD OVERPASS (SITE No. 33-228) WIDENING OF HIGHWAY 7/8 GWP 131-98-00			
TITLE				GRAIN SIZE DISTRIBUTION SAND			
PROJECT No.		08-1132-084-1		FILE No.		0811320841-R030A5	
DRAWN		LMK		Aug 19/10		SCALE N/A REV.	
CHECK						FIGURE A-5	
 Golder Associates LONDON, ONTARIO							

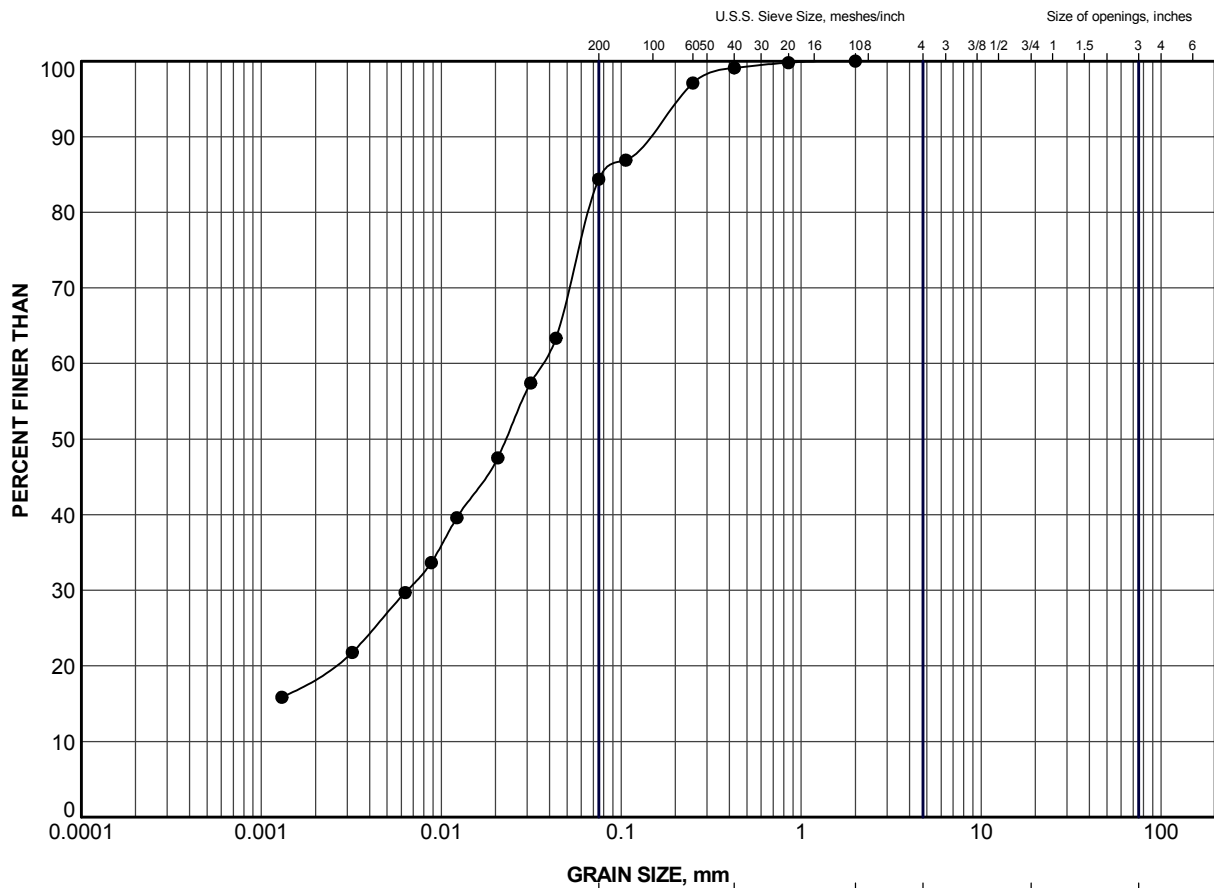


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	303	4	329.9
■	307	14	326.0
▲	308	18	317.0
+	309	11	329.8

PROJECT				WESTMOUNT ROAD OVERPASS (SITE No. 33-228) WIDENING OF HIGHWAY 7/8 GWP 131-98-00			
TITLE				GRAIN SIZE DISTRIBUTION SILTY FINE SAND			
PROJECT No.		08-1132-084-1		FILE No.		0811320841-R030A6	
DRAWN		LMK		SCALE		N/A	
CHECK				REV.			
		Aug 19/10					
 Golder Associates LONDON, ONTARIO				FIGURE A-6			



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

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	302	9	325.7

PROJECT

WESTMOUNT ROAD OVERPASS (SITE No. 33-228)
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE

GRAIN SIZE DISTRIBUTION
SILT



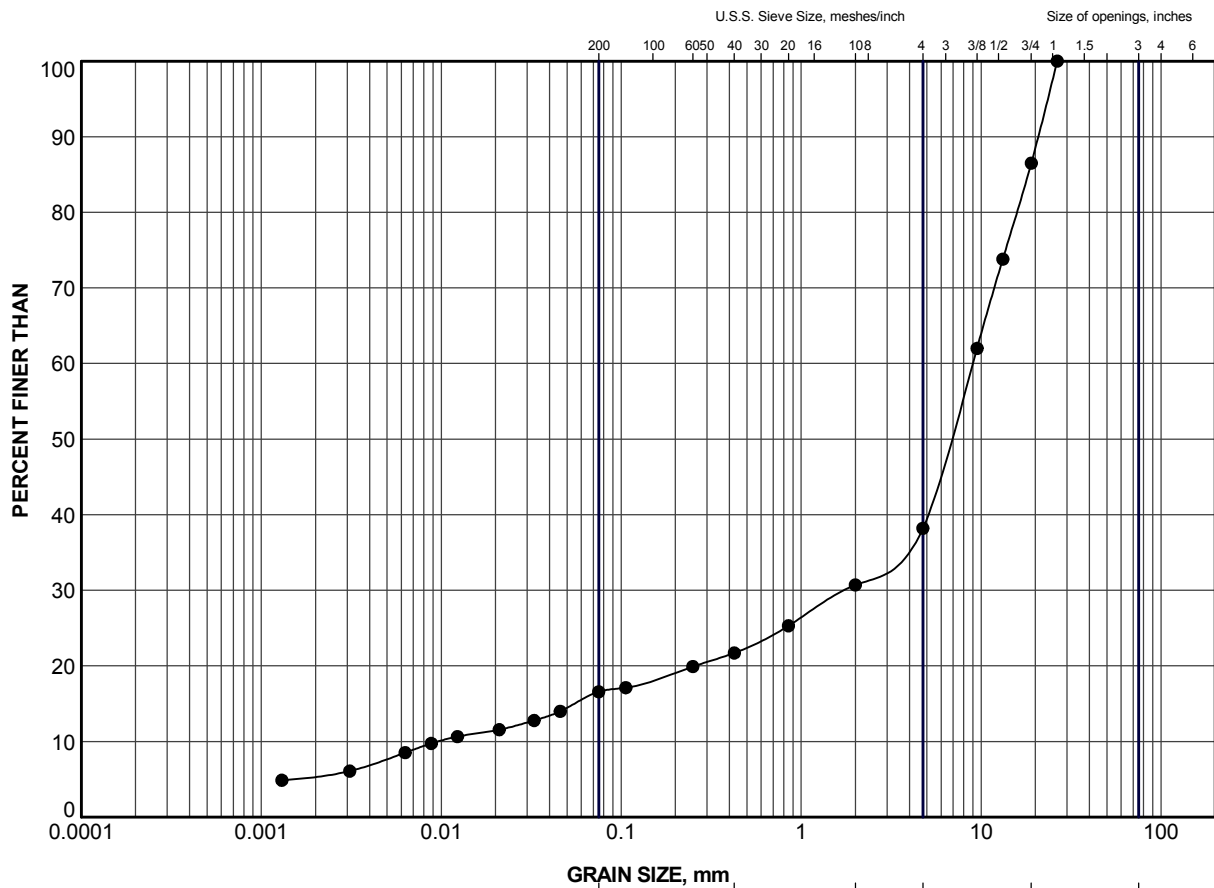
Golder Associates

LONDON, ONTARIO

PROJECT No.	08-1132-084-1	FILE No.	0811320841-R030A7
DRAWN	LMK	Aug 19/10	SCALE N/A REV.
CHECK			

FIGURE A-7

LDN_MTO_NEW_GLDR_LDN.GDT



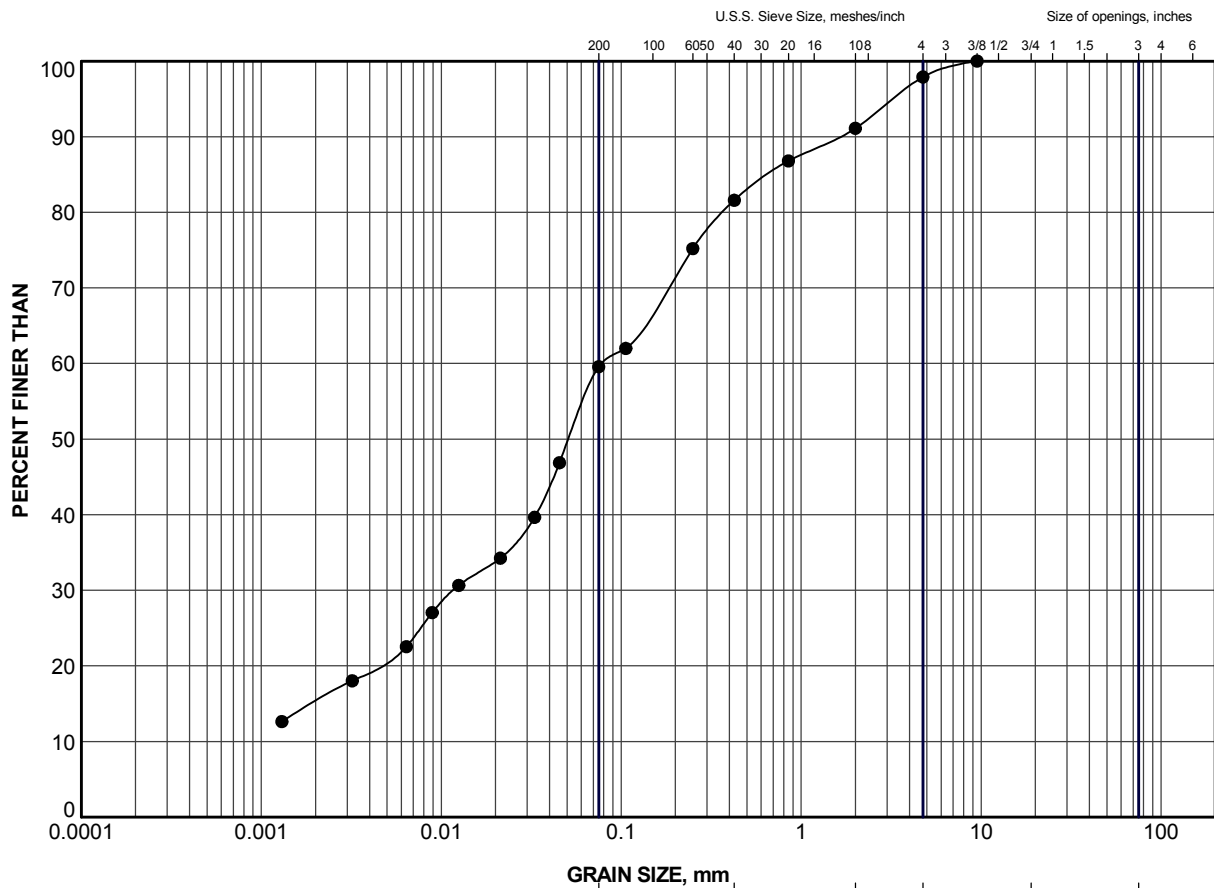
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	308	17	318.5

PROJECT				WESTMOUNT ROAD OVERPASS (SITE No. 33-228) WIDENING OF HIGHWAY 7/8 GWP 131-98-00			
TITLE				GRAIN SIZE DISTRIBUTION SAND AND GRAVEL			
PROJECT No.		08-1132-084-1		FILE No.		0811320841-R030A8	
DRAWN		LMK		SCALE		N/A	
CHECK				REV.			
		Aug 19/10		FIGURE A-8			




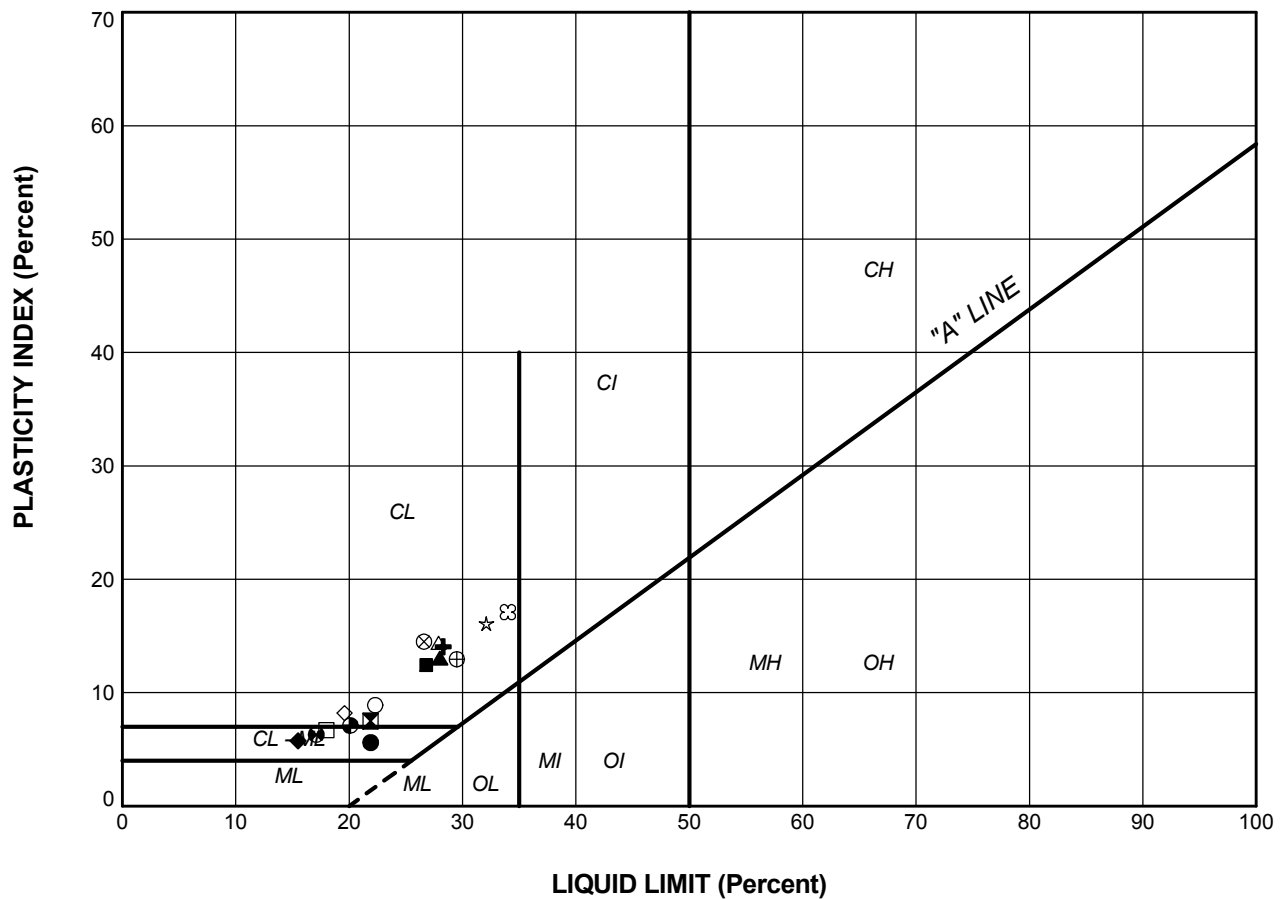


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	302	18	312.0

PROJECT				WESTMOUNT ROAD OVERPASS (SITE No. 33-228) WIDENING OF HIGHWAY 7/8 GWP 131-98-00			
TITLE				GRAIN SIZE DISTRIBUTION SANDY SILT TILL			
PROJECT No.		08-1132-084-1		FILE No.		0811320841-R030A9	
DRAWN		LMK		SCALE		N/A	
CHECK				REV.			
		Aug 19/10					
 Golder Associates LONDON, ONTARIO				FIGURE A-9			



LEGEND

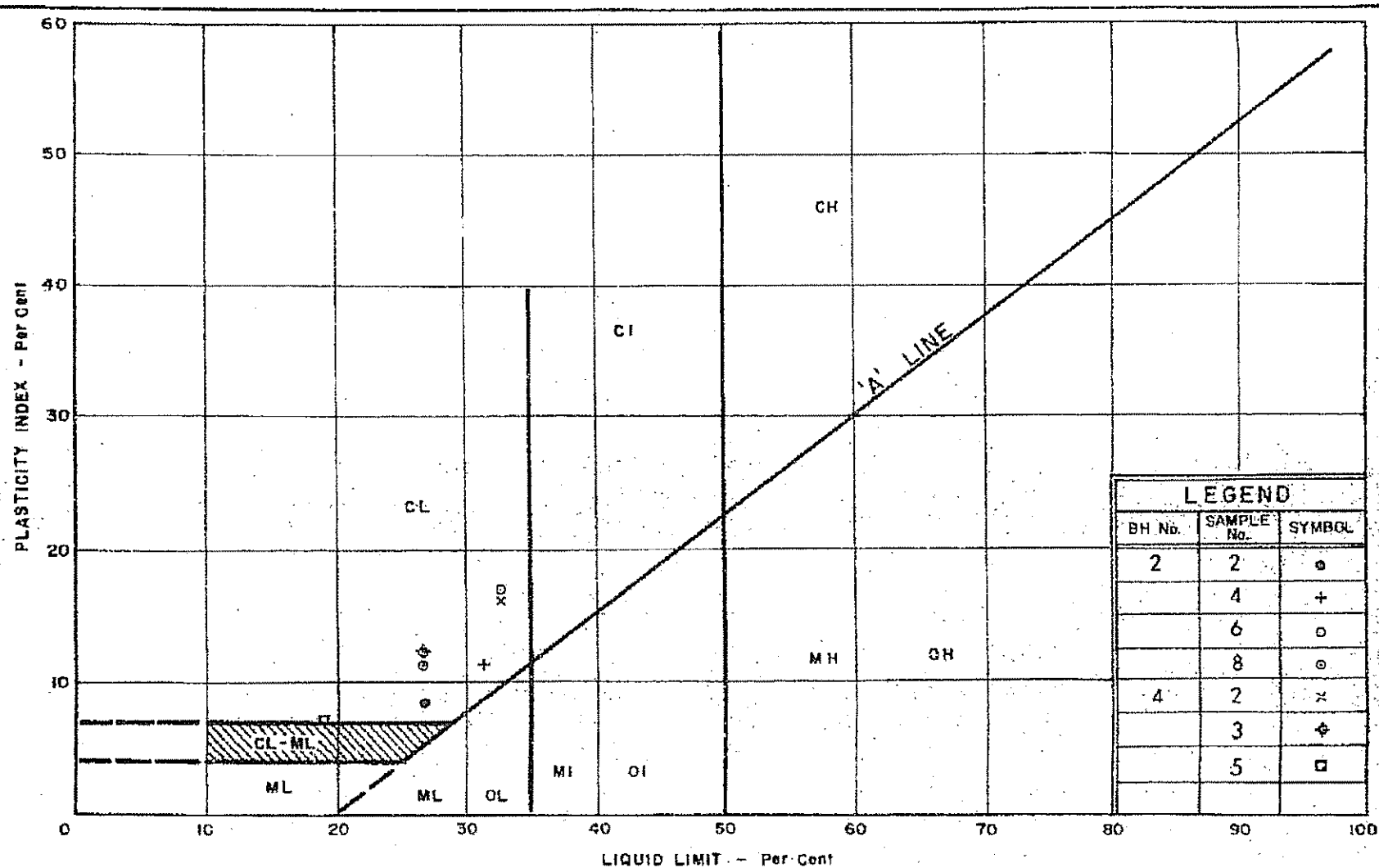
SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
FILL					
●	301	4	21.9	16.3	5.6
○	304	1	22.3	13.4	8.9
⊕	307	9	29.5	16.6	13.0
□	308	3	18.0	11.3	6.7
⦿	309	5	20.1	13.0	7.1
CLAYEY SILT					
▲	301	11	28.0	15.0	13.1
⊕	301	16	28.3	14.3	14.1
△	304	7	27.9	13.5	14.4
⊗	306	14	26.6	12.1	14.5
☆	309	15	32.1	16.0	16.1
⊗	309	17	34.0	16.9	17.1
⊗	310	10	21.9	14.4	7.5
CLAYEY SILT TILL					
■	301	7	26.8	14.4	12.5
◇	302	21	19.6	11.4	8.2
⦿	308	12	17.1	10.8	6.3
SANDY SILT TILL					
◆	302	18	15.5	9.8	5.8

PROJECT			
WESTMOUNT ROAD OVERPASS (SITE No. 33-228) WIDENING OF HIGHWAY 7/8 GWP 131-98-00			
TITLE			
PLASTICITY CHART			
PROJECT No. 08-1132-084-1		FILE No. 0811320841-R030A10	
DRAWN	LMK	Aug 19/10	SCALE N/A REV.
CHECK			
 Golder Associates LONDON, ONTARIO			FIGURE A-10



APPENDIX B

Records of Previous Boreholes (Geocres Report No. 40P08-031)



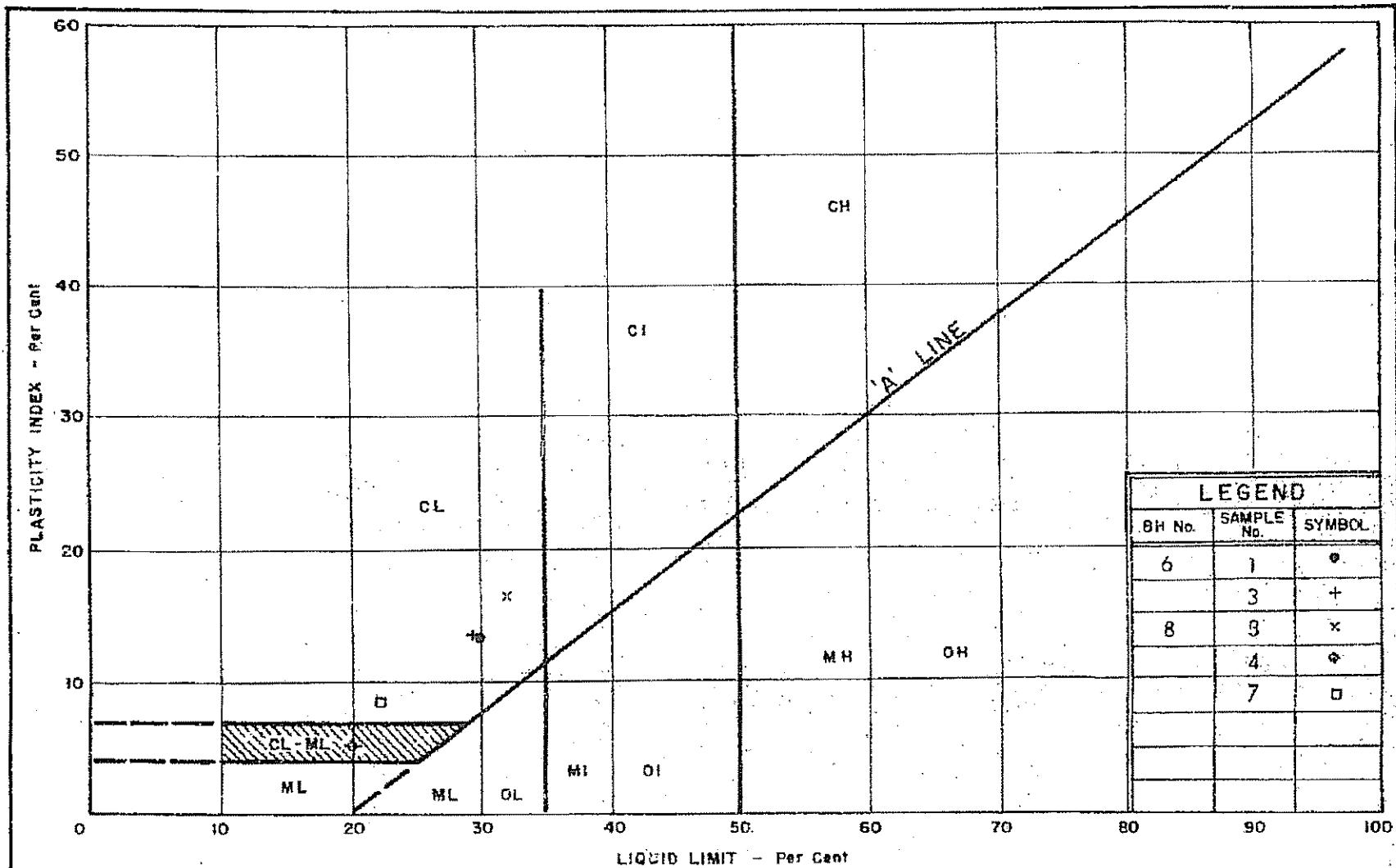
DEPARTMENT OF HIGHWAYS
**MATERIALS and
TESTING
DIVISION**

PLASTICITY CHART **CLAYEY SILT**

WP. No. 628 - 64

JOB No. 67-F-102

Fig. 1a



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART CLAYEY SILT

WP No. 628 - 64

JOB No. 67 - F - 102

Fig. 1b



APPENDIX C

Site Photographs



APPENDIX C

Site Photographs



Photograph 1: South elevation view of Westmount Road overpass.



Photograph 2: North elevation, view of Westmount Road overpass.

n:\active\2008\1132 - geotechnical\1132-000-0\08-1132-084-1 dillon - gwp 131-98-00 fdns - hwy 7-8\reports\0811320841-r03 - westmount rd\0811320841-r03 aug 26 10 (final) appendix c - site photographs.docx

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