**July 2010**

# FOUNDATION INVESTIGATION AND DESIGN REPORT

## Highway 7/8 - Ottawa Street South Overpass Site (33-226)
## 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, Ministry of Transportation - West Region

**Submitted to:**
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REPORT

| | |
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# PART A

# FOUNDATION INVESTIGATION REPORT

HIGHWAY 7/8 - OTTAWA STREET SOUTH OVERPASS
(SITE 33-226)
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
MINISTRY OF TRANSPORTATION - WEST REGION

# 1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the 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 Ottawa Street South (Sites 33-226/01 and 02).

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 - Ottawa Street South overpass is located in the south-central area of Kitchener, Ontario. The site is situated between the Homer Watson Boulevard interchange immediately to the west and the Canadian National Railway spur line to the east. The location of the project is shown on the Key Plan, Figure 1.

The existing overpass consists of two twin structures, each with three spans. This section of Highway 7/8 is currently a four lane divided highway oriented generally east-west. As a result, in addition to the Highway 7/8 main lanes, the interchange eastbound speed change lane and westbound off ramp are also accommodated. The road surface elevation at the structure is at approximate elevation 331.7 metres.

Ottawa Street South is a four lane roadway divided with a concrete median that crosses beneath Highway 7/8 in a north-south orientation through a cut section. Beyond this interchange, Ottawa Street South generally runs east-west. The road surface of Ottawa Street South is at approximate elevation 325.7 metres.

Original grades in the area of the structure varied from elevation 328.5 metres at the west abutment and west pier to 332.6 metres at the east abutment. Adjacent land use is typically urban residential north of Highway 7/8 and industrial/commercial to the south.

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 Hills[1]. 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 Natural Resources Map P.2604 entitled “Quaternary Geology, Cambridge Area, Southern Ontario”, the site lies in an area of primarily ice contact sands deposited in the Pleistocene era. Adjacent to the site, the Maryhill clayey silt till and Port Stanley sandy silt till are 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 Ministry of Natural Resources Map P.1985 entitled “Bedrock Topography Series, Southern Ontario”, the bedrock surface at the site subcrops at about elevation 270 metres or some 55 metres below ground surface.

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

# 3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between October 23 and November 4, 2008, during which time nine 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) |
| 101 | 4 810 293 | 225 366 | 325.52 | 14.11 |
| 102 | 4 810 338 | 225 388 | 331.01 | 11.13 |
| 103 | 4 810 286 | 225 341 | 325.02 | 12.65 |
| 104 | 4 810 332 | 225 336 | 325.93 | 11.07 |
| 105 | 4 810 336 | 225 380 | 331.14 | 29.05 |
| 106 | 4 810 322 | 225 305 | 331.63 | 8.08 |
| 107 | 4 810 281 | 225 308 | 331.72 | 11.13 |
| 108 | 4 810 285 | 225 325 | 331.73 | 20.27 |
| 109 | 4 810 306 | 225 392 | 331.13 | 6.55 |
| | | | | |

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 suitable 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 6.6 and 29.1 metres below the existing pavement or ground surface. Groundwater conditions in the boreholes were observed throughout the drilling operations and piezometers were installed in borehole 101 as indicated on the corresponding Record of Borehole sheets. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 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 labeled 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.

Information from the original geotechnical investigation for the overpass structures was incorporated into this report. Data from boreholes 1 through 4, inclusive, from Geocres Report No. 40P08-051 entitled "Foundation Investigation Report For Ottawa Street South Overpass, Kitchener-Waterloo Expressway, District #4 (Hamilton), W.J. 66-F-69 – W.P. 626-64" dated September 2, 1966 was used to supplement the current data.

The Record of Borehole sheets for previous boreholes 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) |
| 1 | 4 810 322 | 225 322 | 326.26 | 12.65 |
| 2 | 4 810 288 | 225 338 | 326.96 | 12.65 |
| 3 | 4 810 330 | 225 362 | 326.84 | 18.75 |
| 4 | 4 810 296 | 225 374 | 327.54 | 15.24 |

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 encountered the existing pavement structure or topsoil overlying fill materials and a complex sequence of fine grained granular materials which, in turn, are underlain by glacial tills.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profiles, are shown on the attached Drawings 1, 2 and 3. It should be noted that the interpreted stratigraphic profiles have been simplified for clarity. A detailed description of the subsurface conditions encountered in the boreholes is provided on the Record of Borehole sheets and is summarized in the following sections.

## 4.2 Soil Conditions

### 4.2.1 Pavement Structure

Asphalt was encountered at the pavement surface in boreholes 102 and 105 to 109. The asphalt was about 100 to 150 millimetres thick at the borehole locations.

Concrete was encountered beneath the asphalt in borehole 108. The concrete was about 310 millimetres thick at the borehole location.

Pavement granulars were encountered beneath the asphalt or concrete in boreholes 102 and 105 to 109. The granulars were about 150 to 400 millimetres thick with an average thickness of about 300 millimetres.

### 4.2.2 Topsoil and Fill

Layers of topsoil were encountered at the ground surface in boreholes 101, 103 and 104. The topsoil layers were about 150 to 610 millimetres thick with an average thickness of about 350 millimetres.

Fill was encountered beneath the pavement structure or topsoil in all of the boreholes except boreholes 103 and 104. The fill was variable and consisted of sand and gravel to clayey silt materials. The fill materials ranged in thickness from 1.1 to 3.9 metres with an average thickness of about 2.7 metres. The fill had N values, as determined in the standard penetration testing, of 3 to 48 blows per 0.3 metres. Samples of the fill had in situ water contents of 5 to 15 per cent.

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

### 4.2.3 Sand

Layers of compact to very dense sand were encountered beneath the fill in boreholes 101 and 105, beneath the sand and gravel in boreholes 103 and 105, beneath the silty sand in boreholes 102 and 104, beneath the silts in borehole 108 and beneath the sandy silt in borehole 103. These layers were encountered between about elevation 315 and 328 metres. The sands were about 0.6 to 7.2 metres thick where fully penetrated in boreholes 101, 103, 104, 105 and 108 with an average thickness of about 3.3 metres. Boreholes 102 and 103 were terminated in the sands after exploring them for about 6.7 and 2.6 metres, respectively. The sands had N values of 17 to greater than 100 blows per 0.3 metres with natural water contents of about 3 to 20 per cent.

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

A sand layer was found in borehole 4 (Geocres No. 40P08-051) at elevation 318.9 metres beneath the silty sand. This borehole was terminated in a sand layer after exploring some 1.5 metres. The sand was very dense with an N value greater than 100 blows per 0.3 metres and a water content of 14 per cent.

### 4.2.4 Silty Fine Sand and Silty Sand

Compact to very dense silty fine sand was encountered beneath the topsoil in borehole 104, beneath the fill in boreholes 102 and 109, beneath the sands in boreholes 101, 103 and 104, beneath the silts in boreholes 104, 108 and 109, beneath the sandy silt in borehole 108 and beneath the clayey silt in borehole 101. The silty fine sand was encountered between about elevation 312.5 and 329.2 metres. Boreholes 104, 108 and 109 were terminated in silty fine sand after exploring the layers for 0.6 to 2.5 metres. Where fully penetrated, the silty fine sand layers were 0.9 to 4.1 metres thick. The silty fine sand had N values ranging from 10 blows per 0.3 metres to greater than 100 blows per 0.3 metres at depth with natural water contents of about 2 to 10 per cent.

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

Layers of material described as sandy silt to silty sand were encountered at the surface of boreholes 1 to 4 (Geocres No. 40P08-051). For the purposes of this report, this material has been classified as silty sand based on inspection of the reported gradations. Where fully penetrated in boreholes 1, 3 and 4, (Geocres No. 40P08-051) the silty sand layers were 9.1 to 15.2 metres thick. Borehole 2 (Geocres No. 40P08-051) was terminated in the silty sand after exploring it for some 12.7 metres. The silty sand in these boreholes is compact to very dense with N values ranging from 21 to over 100 blows per 0.3 metres and water contents of 6 to 21 per cent.

### 4.2.5 Sandy Silt

Layers of loose to very dense sandy silt were encountered beneath the fill in boreholes 106, 107 and 108, beneath the silty sand and gravel in borehole 108 and beneath the silty fine sand in borehole 103. The sandy silt was encountered between about elevationS 317.3 and 328.0 metres. Borehole 106 was terminated in the sandy silt at about elevation 323.6 metres after exploring it for about 4.4 metres. Where fully penetrated, the sandy silt layers were 0.8 to 5.6 metres thick. The sandy silt had N values of 8 blows per 0.3 metres to greater than 100 blows per 0.3 metres at depth with natural water contents of about 8 to 21 per cent. The sandy silt is of low plasticity based on a single sample with a plastic limit of 11 per cent, a liquid limit of 17 per cent and a plasticity index of 6 per cent. The results of the Atterberg limit determination are shown on Figure A-8.

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

### 4.2.6 Silt

Compact to very dense silt was encountered beneath the topsoil in borehole 103, beneath the sands in boreholes 105 and 108, beneath the silty fine sand in boreholes 104 and 109, beneath the sandy silt in borehole 107 and beneath the clayey silt in 108. The silt was encountered between about elevation 320.7 and 326.0 metres. Borehole 107 was terminated in the silt at about elevation 320.6 metres after exploring it for about 1.1 metres. Where fully penetrated, the silt layers were about 0.4 to 2.1 metres thick. The silt had N values of 21 to 79 blows per 0.3 metres.

A layer described as silt was encountered at elevation 319.3 metres beneath the silty sand in borehole 1 (Geocres No. 40P08-051). This material is considered to be silt based on the reported gradation. The silt is very dense with N values over 100 blows per 0.3 metres and a water content of 17 per cent.

### 4.2.7 Silty Sand and Gravel

Layers of dense to very dense silty sand and gravel were encountered beneath the silt in borehole 105 and beneath the silty fine sand in borehole 108 between about elevation 317.7 and 320.4 metres. These layers were about 0.4 to 0.8 metres thick and had N values of 49 and greater than 100 blows per 0.3 metres.

### 4.2.8 Sand and Gravel

Layers of very dense sand and gravel were encountered beneath the silty fine sand in borehole 101, beneath the silt in borehole 103 and beneath the silty sand and gravel in borehole 105. The sand and gravel was encountered between about elevation 315.2 and 323.5 metres. Borehole 101 was terminated in the sand and gravel at about elevation 311.4 metres after exploring it for about 3.8 metres. Where fully penetrated in boreholes 103 and 105, the sand and gravel layers were about 0.8 and 1.5 metres thick, respectively. The sand and gravel had N values of 58 to greater than 100 blows per 0.3 metres with natural water contents of about 11 to 16 per cent.

Grain size distribution curves for samples of the sand and gravel recovered from the standard penetration testing in boreholes 101 and 105 are provided on Figure A-5.

### 4.2.9 Clayey Silt

Firm to very stiff clayey silt was encountered beneath the silty fine sand in borehole 101 and beneath the sandy silt in borehole 108. The clayey silt was encountered at about elevations 318.2 and 326.6 metres in boreholes 101 and 108, respectively. The clayey silt was about 1.4 to 2.8 metres thick at the borehole locations. The clayey silt had N values of 9 to 7 blows per 0.3 metres.

A grain size distribution curve for a sample of the clayey silt recovered from the standard penetration testing in borehole 107 is provided on Figure A-6.

### 4.2.10 Silty Clay Till

Beneath the sands, borehole 105 encountered a 9.3 metre thick layer of hard silty clay till at about elevation 315.3 metres. The silty clay till had N values of 62 to 103 blows per 0.3 metres with natural water contents of about 15 to 24 per cent. The silty clay till had average plastic and liquid limits of 19 and 49 per cent, respectively, based on two Atterberg limits determinations. These data are provided on Figure A-9 and indicate that the silty clay till is of medium plasticity.

Grain size distribution curves for samples of the silty clay till recovered from the standard penetration testing in borehole 105 are provided on Figure A-7. Although not specifically encountered in the boreholes, the presence of cobbles and boulders in the glacial till should be expected.

A deposit classified as silty clay to clayey silt was encountered at elevation 314.5 metres below the silty sand in borehole 3 (Geocres No. 40P08-051). This material is interpreted to be silty clay till for the purpose of this report. The silty clay till is hard with N values of 98 to over 100 blows per 0.3 metres and water contents of 14 and 17 per cent.

### 4.2.11 Sandy Silt Till

Beneath the silty clay till, very dense sandy silt till was encountered in borehole 105 at about elevation 306.0 metres. Borehole 105 was terminated in the sandy silt till at about elevation 302.1 metres after exploring it for about 3.9 metres. Due to the nature of glacial till deposits, cobbles and boulders may be present in the sandy silt till although not specifically encountered in the boreholes. The sandy silt till had N values in excess of 100 blows per 0.3 metres with a natural water content of about 9 per cent.

A grain size distribution curve for a sample of the sandy silt till recovered from the standard penetration testing in borehole 105 is provided on Figure A-8.

## 4.3 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling and piezometers were installed in borehole 101. Installation details are provided on Record of Borehole 101 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) | | Measured Groundwater Elevation (m) | | |
|---|---|---|---|---|---|---|
| | | | Installation | October 24, 2008 | August 25, 2009 | June 3, 2010 |
| 101 | 325.52 | 321.7 | Shallow Piezometer<br>Deep Piezometer | 321.77<br>321.31 | 321.71<br>321.43 | 321.20<br>320.74 |
| 102 | 331.01 | Dry | - | - | | |
| 103 | 325.02 | 321.2 | - | - | | |
| 104 | 325.93 | 319.8 | - | - | | |
| 105 | 331.14 | * | - | - | | |
| 106 | 331.63 | Dry | - | - | | |
| 107 | 331.72 | 324.3 | - | - | | |
| 108 | 331.73 | * | - | - | | |
| 109 | 331.13 | * | - | - | | |
| | | | | | | |
| 1 (40P08-051) | 328.45 | 319.5 | - | - | | |
| 2 (40P08-051) | 329.03 | 319.9 | - | - | | |
| 3 (40P08-051) | 329.70 | 319.4 | - | - | | |
| 4 (40P08-051) | 332.57 | 321.0 | - | - | | |

* Groundwater level not established.

Boreholes 102, 106 and 109 remained dry during drilling. The groundwater level in boreholes 105 and 108 could not be established as rotary drilling methods were employed to advance these boreholes. Groundwater was encountered in the other boreholes at depths of 3.8 to 7.5 metres or between elevation 319.8 and 324.3 metres.

Piezometers were installed in borehole 101. A shallow piezometer was installed within the surficial sands and a deep piezometer was installed in the sand and gravel. On June 3, 2010, the water level in the shallow piezometer was about 4.3 metres below ground surface or at about elevation 321.2 metres.  The water level in the deep piezometer was about 4.8 metres below ground surface or at about elevation 320.7 metres.

Groundwater was encountered in previous boreholes 1 to 4 between elevations 319.5 to 321.0 metres or depths of 8.7 to 11.6 metres.

Based on the measured and encountered groundwater levels, the inferred groundwater level is at elevation 322 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.

## 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 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 Project Manager, Mr. Philip R. Bedell, P.Eng. This report was reviewed by Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

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# PART B

# FOUNDATION DESIGN REPORT

HIGHWAY 7/8 - OTTAWA STREET SOUTH OVERPASS
(SITE 33-226)
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

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 Highway 7/8 - Ottawa Street South overpass widening and rehabilitation 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.

## 6.2 Existing Structure

The Ottawa Street (South) overpass was built in 1967. Information on the existing structure was obtained from a review of the original request for proposal, Department of Highways Ontario (DHO) Drawing No. D-6056-1 entitled “General Arrangement: Kitchener-Waterloo Expressway, Ottawa Street South Overpass” dated July 1967, DHO Drawing No. D-6056-3 entitled “Footing & Pile Layout & Details” dated July 1967” and Geocres Report No. 40P08-051. The overpass structure is comprised of twin three span concrete structures with post tensioned decks. The overall overpass structure is approximately 55.4 metres long on the north side and 56.0 metres on the south side. The overall width varies from 38.5 metres on the east side to 43.7 metres on the west side.

The original design information indicates that the bridge piers are supported by spread footings 2.74 metres wide by 34 to 36.5 metres long founded at about elevation 324 metres. The geotechnical report for this site recommended an allowable bearing pressure of 290 kilopascals (kPa) for the footings (Geocres No. 40P08-51). The abutments are founded on concrete filled steel tube piles having a outer diameter of 324 millimetres and wall thickness of 6.3 millimetres. The piles were designed to be 4.9 metres long with a working stress design load of 600 kilonewtons (kN). The piles were to be driven to about elevation 323.9 metres. The existing embankments are up to approximately 4.5 metres high.

### 6.2.1 Geotechnical Resistances for Existing Foundations

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

| | | | | | | **Geotechnical Resistances** | |
|---|---|---|---|---|---|---|---|
| **Location** | **Foundation Type** | **Cut-Off Elevation** | **Pile Tip or Underside of Footing Elevation** | **Pile Length** | **Founding Strata** | **Factored ULS** | **SLS** |
| | | **(m)** | **(m)** | **(m)** | | | |
| West abutment | Tube Pile | 329.11 | 323.23 | 4.88 | Dense to very dense sandy silt/silty sand/silt | 900 kN | 600 kN |
| West pier | Spread footing | - | 323.55 | - | Dense sandy silt/silty sand to very dense sand and gravel/silt | 450 kPa | 300 kPa |
| East pier | Spread footing | - | 323.85 | - | Very dense sandy silt/silty sand to dense sand | 450 kPa | 300 kPa |
| East abutment | Tube Pile | 328.73 | 323.85 | 4.88 | Very dense sandy silt/silty sand to dense to very dense sand | 900 kN | 600 kN |

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.

The lateral geotechnical resistance for the existing 324 millimetre O.D. steel tube piles can be taken as 150 kilonewtons (factored) at ULS and 65 kilonewtons at SLS. The SLS value corresponds to a lateral deflection of 10 millimetres.

### 6.2.2 Conversion to Semi-Integral Abutments

The existing conventional abutments to semi-integral abutments are to be converted to semi-integral abutments. The configuration of the existing piled abutment foundation features two rows of steel tube piles. One row of vertical piles is spaced approximately 4.57 and 4.45 metres apart at the west and east abutment respectively. The second row of piles features piles inclined at 6 vertical to 1 horizontal metres and spaced 2.37 metres apart at the west abutment and 2.32 metres at the east abutment. The existing foundations and subsurface conditions are considered to be compatible with a semi-integral abutment design.

## 6.3 Proposed Work

A Preliminary General Arrangement Drawing for the Ottawa Street South Overpass was provided by Dillon in digital format in June 2010. According to this drawing, the existing structure will be widened to accommodate an additional lane in the westbound and eastbound through lanes and an additional lane along the Ottawa Street South E-N/S Ramp. The existing deck and parapet walls will be removed and replaced. The existing pier columns will be modified. The approach slab at the west abutment will be replaced and the existing abutments will be converted from conventional to semi-integral abutments. The abutment conversion will require installation of temporary roadway protection.

The grade in this area will be raised between 0.7 to 1.0 metres in this area. On the south side of the structure, the approach embankments will be widened about 2.5 metres on the west side and 4.0 metres on the east side. Approach embankment widening on the north side of the structure will vary between 5.0 metres on the west side and 10 metres on the east side. Installation of a barrier wall and a Retained Soil System (RSS) wall is proposed for the northeast quadrant of the structure.

It is proposed to support the widened piers on shallow footings at elevations matching the existing pier footings. The widened abutments will be supported on tube piles with top of pile cap elevations of 329.57 and 329.18 metres for the west and east abutments, respectively.

## 6.4 Bridge Widening Foundations

The subsoils encountered in the boreholes put down during the investigation typically consist of surficial fills over a complex and variable sequence of sand, sand and gravel, sandy silt, silty sand, clayey silt, and glacial till. Based on a review of the standard penetration test profile of the boreholes, it was generally noted that very dense or hard soils with N values greater than 100 blows per 0.3 metres were present closer to the surface on the north side of structure than on the south side. The prevailing groundwater level was inferred to be at approximately elevation 322 metres.

Based on the results of the boreholes and the existing bridge foundations, it is recommended that deep foundations be used for the widened abutments and shallow foundations for the east and west pier widenings in order to reduce the magnitude of differential settlement between the existing and proposed structures and to facilitate construction. Alternatively, the widened abutments could be supported on spread footings. However, shallow foundations may not provide sufficient geotechnical resistance for the proposed widening at the south end of the west abutment and the risk of differential settlements is higher than deep foundations. Deep foundations are not considered warranted for support of the piers.

A comparison of foundation alternatives is presented in Table 1. The cost provided are estimates meant to provide an order of magnitude comparison amongst the alternatives and are not indicative of actual construction costs.

### 6.4.1 Deep Foundations

The abutments for the proposed widenings can be designed using driven HP 310 x 110 steel H-piles or 323 millimetre outer diameter (O.D.), concrete filled steel tube piles with a nominal 9.5 millimetre thick wall thickness. The preferred technical alternative for the abutment foundations is tube piles. Deep foundations can also be utilized for support of the piers. However since competent bearing layers are present near the surface, the expense and time to construct deep foundations at the pier locations is not considered warranted.

### *Geotechnical Axial Resistance – Driven Steel H-Piles*

For design, the factored axial geotechnical resistances at Ultimate Limit States (ULS) for HP 310 x 110 piles driven to refusal into the very dense deposits at or below the elevations shown in the following table. The SLS values assume 25 millimetres of settlement. For the purposes of deails design, it was assumed that the cut-off elevations will be similar to those used for the existing structure.

| | | | | | Geotechnical Resistances | |
|---|---|---|---|---|---|---|
| **Location** | **Cut-Off Elevation** | **Proposed Tip Elevation** | **Pile Length** | **Founding Strata** | **Factored ULS** | **SLS** |
| | **(m)** | **(m)** | **(m)** | | **(kN)** | **(kN)** |
| West abutment-north side | 329.1 | 321.5 | 7.6 | Very dense sandy silt | 900 | 750 |
| West abutment-south side | 329.1 | 315.0 | 14.1 | Very dense sandy silt | 1350 | 1150 |
| East abutment – north side | 328.7 | 314.3 | 14.4 | Hard silty clay till | 1500 | 1250 |
| East abutment – south side | 328.7 | 320.0 | 8.7 | Very dense sand | 1250 | 1000 |

The steel H-piles should be installed and monitored in accordance with OPSD 3000.150 and SP903S01. The piles are to be equipped with reinforced flanges as shown in Ontario Provincial Standard Drawing (OPSD) 3000.100

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

A pile note is to be added to the foundation drawing that states that piles 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 above table 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.

### *Geotechnical Axial Resistance – Driven Steel Tube Piles*

Concrete filled steel tube piles 323 millimetre O.D. x 9.5 millimetre wall thickness driven closed ended may be used for support of the widened embankments for modest loadings. A factored ULS resistance of 900 kilonewtons and a geotechnical resistance of 600 kilonewtons at SLS is available for tube piles driven to or below the elevations shown in the following table. The SLS values assume 25 millimetres of settlement.

For the purposes of detail design, it was assumed that the cut-off elevations will be similar to those used for the existing structure.

| Location | Cut-Off Elevation | Proposed Tip Elevation | Pile Length | Founding Strata |
|---|---|---|---|---|
| | (m) | (m) | (m) | |
| West abutment-north side | 329.1 | 324.3 | 4.8 | Very dense sandy silt |
| | | | | |
| West abutment-south side | 329.1 | 317.0 | 12.1 | Very dense sandy silt |
| | | | | |
| East abutment – north side | 328.7 | 320.0 | 8.7 | Very dense silty sand and gravel |
| | | | | |
| East abutment – south side | 328.7 | 323.4 | 5.2 | Very dense silty sand |

The piles are to be equipped with reinforced flanges as shown in OPSD 3001.100. The steel tube piles should be installed and monitored in accordance with OPSD 3001.150 and SP903S01.

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

A pile note is to be added to the foundation drawing that states that piles 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 above table. The wording of the pile note should match Note 1 of Section 3.3.3 of the MTO Structural Manual.

### *Construction Considerations*

It should be noted that cobbles and boulders may be present in the till soils and may impact pile driving operations. Hard driving may be experienced in the very dense sand and sand and gravel layers below elevation 327 metres at the northeast abutment widening and near elevation 326 metres at the southeast abutment widening. A non-standard special provision (NSSP) should be added to the contract documents to alert the contactor to the presence of cobbles and boulders within till soils and zones where hard driving may occur.

### *Downdrag Load (Negative Skin Friction)*

A relatively low grade raise of 1.1 metres will be carried out that the approach embankments in conjunction with the proposed widening. Limited fill placement is anticipated at the east embankment, particularly in the abutment area since the pre-construction grades were such that the embankment in this area was constructed in a cut. Approximately 20 metres behind the east abutment, the fill depth is currently up to 3.2 metres. However fills up to 5.6 metres will be required at the west embankment. Considering the relatively low grade raise and the predominantly compact to very dense cohesionless soils, negligible negative skin friction is expected to develop on the existing and new piles at both abutments.

Any potential downdrag loads can be reduced or eliminated by installing the piles well after the fill has been placed.

## 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, inclined piles must provide the resistance to the lateral loading. The stratigraphy presented in the table below has been simplified for the purpose of this report. The horizontal reaction to the pile can be estimated using the following equation and ranges in subgrade reaction coefficient where:

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

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

d = pile width or diameter (m)

$n_h$, $k_{Si}$ = constant of horizontal subgrade reaction (MPa/m)

z = depth below ground surface grade (m)

| Soil Type | Elevation (m) | | $n_h$ | $S_u$ |
|---|---|---|---|---|
| | From | To | (MPa/m) | (MPa) |
| Loose to compact fill (sandy silt) | Surface | 327 | - | - |
| Stiff clayey silt (west abutment only)<br>Compact to very dense sand/compact sandy silt/compact to dense silt/compact sand | 327<br>327 | 325<br>320 | -<br>5 – 10 | 0.06 – 0.09<br>- |
| Dense to very dense sand and gravel/silty fine sand | 320 | 317 | 10 - 12 | - |
| Very dense sand (east abutment only)<br>Very dense sandy silt/silty fine sand (west abutment only) | 317<br>317 | 315<br>311 | 10 – 12<br>10 – 12 | -<br>- |
| Hard silty clay till (east abutment only) | 315 | 306 | - | 0.42 – 0.58 |
| Very dense sandy silt till (east abutment only) | 306 | 302 | 10 – 12 | - |

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

| ***Pile Spacing in Direction of Loading, d = Pile Diameter*** | ***Subgrade Reaction Reduction Factor R*** |
|---|---|
| 8d | 1.00 |
| 6d | 0.70 |
| 4d | 0.40 |
| 3d | 0.25 |

For the purpose of design, the lateral geotechnical resistance for an HP 310 x 110 pile can be taken as 115 kilonewtons at factored ULS and 45 kilonewtons at SLS based on assessed values quoted in Table C6.4 of the CHBDC. For a 323 millimetre diameter, 9.5 millimetre wall thickness tube piles, the corresponding resistances are 170 kN at factored ULS and 125 kN at SLS based on Brom's Method. The SLS values are based on 10 millimetres of deflection.

### *Frost Protection*

The pile caps should be provided with a minimum of 1.4 metres of soil cover for frost protection or thermal equivalent.

### *Monitoring of Existing Structure*

The process of installing the new piles will produce ground vibrations in the surrounding soils. It is anticipated that structures more than one pile length away from the areas where the new piles are constructed are not likely to be affected[2]. The overpass structure will be widened no more than approximately 4 metres at any side of each abutment. Loose, clean, saturated and uniform granular soils were considered to be the most problematic with respect to pile driving vibration. Such soils were not encountered in the boreholes. Assuming that the pile spacing will be similar to that of the existing structure, a small number of piles will be installed. Therefore, vibration and settlement monitoring in conjunction with the pile driving operations is not considered warranted unless the existing structure is in a condition that it is susceptible to vibrations or the existing buried utilities immediately adjacent to the structure are sensitive to vibrations.

### 6.4.2 Shallow Foundations

The existing west and east piers of the Ottawa Street South overpass structure are supported on spread footings. The existing structure appears to be performing adequately with the piers founded on shallow foundations. Spread footings are the preferred technical alternative for support of the piers considering the shallow depth to competent soils, the lower cost and duration of construction and the avoidance of vibration related damage due to pile driving. The closest structure is approximately 40 metre away in the northwest quadrant.

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[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

## Geotechnical Resistance

Assuming the footings for the widened piers are constructed using the same dimensions and founded at the same depth as those of the existing piers, the following geotechnical resistances can be used for design:

| | | | | **Geotechnical Resistances** | |
|---|---|---|---|---|---|
| **Location** | **Underside of Footing Elevation** | **Inferred Groundwater Elevation** | **Founding Strata** | **Factored ULS** | **SLS** |
| | **(m)** | **(m)** | | **(kPa)** | **(kPa)** |
| West pier – north side | 323.55 | 322 | Very dense silt to silty fine sand | 600 | 450 |
| West pier – south side | 323.55 | 322 | Compact to dense silty fine sand/very dense sand and gravel/sand | 450 | 300 |
| East pier – north side | 323.85 | 322 | Very dense sandy silt/silty sand | 600 | 450 |
| East pier – south side | 323.85 | 322 | Dense sand | 450 | 300 |

The SLS resistances are based on 25 millimetres of settlement. Construction staging will reduce post construction differential settlements.

## Resistance to Lateral Forces

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

| | | |
|---|---|---|
| Footings on dense silt | angle of friction | $30^0$ |
| | tan δ | 0.58 |
| Footings on very dense sand and gravel | angle of friction | $35^0$ |
| | tan δ | 0.70 |
| Footings on very dense sandy silt or dense sand | angle of friction | $32^0$ |
| | tan δ | 0.62 |

### *Frost Protection*

All footings should be provided with a minimum of 1.4 metres of earth cover or thermal equivalent for frost protection purposes.

### *Construction Considerations*

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

## 6.5 Liquefaction Potential and Seismic Analysis

### 6.5.1 Seismic Parameters

The site is located in Kitchener, 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 (SPZ) 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 lifeline bridge. Multi-span bridges situated in SPZ 1, such as the subject structure, need not be analyzed for seismic loads. However design forces for restraining elements and bridge support lengths must meet the minimum requirements as outlined in CHDBC 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 topsoil overlying an extensive deposit of compact to very dense sands, silts and sand and gravel with interlayers of clayey silt. Silty clay till and sandy silt till were encountered below elevation 315 metres. 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 55 metres or below approximate elevation 270 metres. Based on the site stratigraphy, the soil profile type is categorized as Type I with a seismic site response coefficient, S of 1.0 based on the CHBDC criteria.

### 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 0.005 millimetres are present below the groundwater table, these layers generally 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 date to the Pleistocene era. Deposits from the Pleistocene era historically have a very low to low susceptibility to liquefaction upon strong ground shaking. Therefore the liquefaction potential is considered to be relatively low based on the soil profile type, age of the deposits, relative density and the historically low seismicity. Therefore a detailed evaluation of the liquefaction potential of the foundation soils, impact of liquefaction on the bridge foundations and the effect of seismic forces on embankment stability is not considered warranted unless the structure is deemed to be a lifeline bridge.

## 6.6 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments 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 SP 105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with Ontario Provincial Standard Drawing (OPSD) 3101.150 and 3190.100.
- A compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with CHBDC, Figure 6.6. Compaction equipment should be used in accordance with SP 105S10.
- The granular fill may be placed either in a zone with a width equal to at least 1.4 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).

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[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.

- 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/m³ |
| 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/m³ | 21 kN/m³ |
| Coefficients of lateral earth pressure: | | |
| Active, $K_a$ | 0.27 | 0.31 |
| At rest, $K_o$ | 0.43 | 0.47 |

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.
- If semi-integral abutment design allows for movement of the bridge deck ends, passive earth pressures may be used in the geotechnical design of the structure. The movements required to fully mobilize 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. The lateral earth pressure coefficients should be adjusted if there is sloping ground at the back of the wall.

## 6.7 Embankments

It is understood the existing embankment will undergo a one metre high grade raise increasing the maximum height from approximately 4.5 metres to 5.5 metres at the west embankment , and 3.2 to 4.2 metres at the east embankment. As noted previously, the east embankment in the vicinity of the abutment was constructed in a cut. The width of the crest widening will be approximately 5 metres on the north side and 3 metres on the south side. The fill materials are to consist of well compacted on site borrow materials or imported granular fills.

### 6.7.1 Subgrade Preparation and Embankment Construction

All surficial topsoil, organic, loose, soft and/or otherwise deleterious materials should be stripped from areas of proposed embankment widenings. The exposed subgrade should be proofrolled prior to fill placement under the direction of qualified geotechnical personnel. Grading and embankment construction should be conducted in accordance with MTO Special Provision 206S03.

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 Ontario Provincial Standard Drawing (OPSD) 208.010 and compacted. Upon completion of filling to the pavement subgrade level, the embankment side slopes 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. A post-construction settlement criteria recommended by other MTO jurisdictions 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 in all quadrants except in the southwest quadrant and at the centreline of both approach embankments. Noting the relatively low grade raise, limited width of the widening areas and presence of relatively deep deposits of predominantly compact to very dense granular foundation soils, 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. At the southwest quadrant where a 2 metre thick clayey silt layer is present below elevation 326.6 metres, somewhat larger settlements are expected in the widened areas as summarized in the following table:

| **Location** | **Estimated Total Settlement (mm)** | | |
|---|---|---|---|
| | **Crest of widened embankment** | **Toe of existing embankment** | **Toe of widening embankment** |
| Southwest widening – abutment | 30 | 20 | 45 |
| Southwest widening – 30m behind abutment | 45 | 50 | 40 |

It is anticipated that at least 60 per cent of the settlement in the southwest quadrant will occur during construction. Therefore post-construction settlement within the travelled area of the widened embankment is expected to be within the MTO's criteria.

### 6.7.3 Stability

Embankment side slopes formed no steeper than 2 horizontal to 1 vertical are considered suitable for this site. A Factor of Safety against deep seated failure of greater than 1.3 is available for embankments constructed with the native materials founded on the compact to very dense granular soils prevalent at this site.

## 6.8 Excavations and Temporary Cut Slopes

Excavations for pile caps and spread footing construction will extend primarily through the existing fill materials, and surficial compact to very dense sands and silts. Excavations for spread footings for the piers and pile caps for the abutments are not expected to encounter the groundwater level which has been inferred at elevation 322 metres. Temporary open cut slopes within the fill materials should be maintained no steeper than 1 horizontal to 1 vertical. Groundwater control such as pumping from properly constructed and filtered sumps may be required based on timing of construction and prevailing weather conditions.

The consideration with respect to protection of the founding soils, however, as given in Section 6.2.2 under the heading Construction Considerations must be recognized. Sumps should be maintained outside of the actual footing limits. Surface water runoff should be directed away from the excavations at all times. The appropriate Non Standard Special Provision (NSSP) should be included in the contract documents.

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, properly dewatered cohesionless materials, and glacial tills would be classified as Type 2 soils.

### 6.8.1 Temporary Roadway Protection

Widening and raising the grade of the existing approach embankments will require installation of temporary roadway protection systems as the construction will occur in stages. The initial stage (Stage 1) consists of preparatory work in the westbound 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 whilst the southern embankment will be widening and the grade raise, and the southern side of the overpass structure will be rehabilitated and widening (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).

It has been proposed to install temporary roadway systems in the median and along the crest of the existing embankments. 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. Temporary support systems could consist of soldier piles and lagging where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds or driven steel sheet piling. Support to the systems could be in the form of struts and walers in the case of footing excavations or rakers and anchors in the case of roadway protection. The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area line or point loads as well as the impact of sloping ground behind the system.

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.

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

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

For the granular fill and native 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
$\gamma$ = soil unit weight
$q$ = surcharge for traffic and other loading

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

$p$ = $0.2\gamma H_T$ to $0.4\gamma H_T$
where $H_T$ = the total height of the excavation
$\gamma$ = 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 | | | ANGLE OF INTERNAL FRICTION | UNIT WEIGHT |
|---|---|---|---|---|---|
| | ACTIVE, $K_a$ | At Rest, $K_o$ | Passive, $K_p$ | | |
| | | | | (degrees) | ($kNm^{-3}$) |
| | | | | | |
| Granular Fill | 0.38 | 0.55 | 2.7 | 27 | 18.5 |
| Cohesive Fill | 0.41 | 0.58 | 2.5 | 25 | 18.5 |
| Sandy Silt | 0.35 | 0.52 | 2.9 | 29 | 19.0 |
| Clayey Silt | 0.38 | 0.55 | 2.7 | 27 | 19.0 |
| Sand | 0.33 | 0.50 | 3.0 | 20 | 20.0 |
| Silty Sand/Silty Fine Sand | 0.32 | 0.48 | 3.1 | 31 | 20.0 |
| Silt | 0.33 | 0.50 | 3.0 | 30 | 20.0 |

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 temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 105S19.

# 7.0 MISCELLANEOUS

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

**GOLDER ASSOCIATES LTD.**

| | |
|---|---|
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TABLE I

## COMPARISON OF FOUNDATION ALTERNATIVES

Site 33-226
Highway 7/8 - Ottawa Street South Overpass
Highway 7/8 Widening
GWP 131-98-00

| FOUNDATION OPTION | FEASIBILITY | ADVANTAGES | DISADVANTAGES | ESTIMATED COSTS | RISKS/ CONSEQUENCES |
|---|---|---|---|---|---|
| Spread footings supported on dense to very dense native granular soils | • Feasible<br>• Preferred technical alternative for pier foundations | • Least expensive option<br>• Ease of construction | • May provide insufficient capacity for abutment loads<br>• Possibility of differential settlement between widened and pre-existing areas if used for abutment foundations | • Estimated cost $17,000 per pier<br>• Less expensive than deep foundation options | • Relatively low risk |
| End bearing steel H-pile foundations driven to refusal into “100+ blow” native granular materials or silty clay till | • Feasible | • High bearing resistance<br>• Negligible settlement<br>• Less vibration related damage compared to steel tube piles | • Differential performance than existing tube pile<br>• Higher capacity than steel tube piles when driven to end bearing | • Estimated cost $26,500 per abutment<br>• More expensive than shallow foundations; cost competitive with tube piles | • Possible pile tip damage if piles are not adequately protected while driving through very dense/hard soils<br>. |

## COMPARISON OF FOUNDATION ALTERNATIVES

| FOUNDATION OPTION | FEASIBILITY | ADVANTAGES | DISADVANTAGES | ESTIMATED COSTS | RISKS/ CONSEQUENCES |
|---|---|---|---|---|---|
| End bearing or friction concrete filled steel tube piles driven to into very dense native granular materials | • Feasible<br>• Preferred technical option | • High bearing resistance<br>• Negligible settlement | • More costly than shallow footings | • Estimated cost $25,000<br>• More expensive than shallow foundations | • Possible pile tip damage if piles are not adequately protected while driving through very dense/hard soils |

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 |

## II. PENETRATION RESISTANCE

**Standard Penetration Resistance (SPT), N:**

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

**Dynamic Cone Penetration Resistance; $N_d$:**

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

**PH:** Sampler advanced by hydraulic pressure
**PM:** Sampler advanced by manual pressure
**WH:** Sampler advanced by static weight of hammer
**WR:** Sampler advanced by weight of sampler and rod

**Piezo-Cone Penetration Test (CPT)**

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

## III. SOIL DESCRIPTION

### (a) Cohesionless Soils

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

### (b) Cohesive Soils

| Consistency | $c_u,s_u$ | |
|---|---|---|
| | kPa | psf |
| Very soft | 0 to 12 | 0 to 250 |
| Soft | 12 to 25 | 250 to 500 |
| Firm | 25 to 50 | 500 to 1,000 |
| Stiff | 50 to 100 | 1,000 to 2,000 |
| Very stiff | 100 to 200 | 2,000 to 4,000 |
| Hard | over 200 | over 4,000 |

## IV. SOIL TESTS

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

| | |
|---|---|
| $\pi$ | 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

| | |
|---|---|
| $\gamma$ | shear strain |
| $\Delta$ | change in, e.g. in stress: $\Delta\ \sigma$ |
| $\varepsilon$ | linear strain |
| $\varepsilon_v$ | volumetric strain |
| $\eta$ | coefficient of viscosity |
| v | poisson's ratio |
| $\sigma$ | total stress |
| $\sigma'$ | effective stress ($\sigma' = \sigma$-u) |
| $\sigma'_{vo}$ | initial effective overburden stress |
| $\sigma_1, \sigma_2, \sigma_3$ | principal stress (major, intermediate, minor) |
| $\sigma_{oct}$ | mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$ |
| $\tau$ | 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 |
| $\gamma'$ | 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 |
| $w_1$ | liquid limit |
| $w_p$ | plastic limit |
| $l_p$ | plasticity index = $(w_1 - w_p)$ |
| $w_s$ | shrinkage limit |
| $I_L$ | liquidity index = $(w - w_p)/I_p$ |
| $I_C$ | consistency index = $(w_1 - w)\ /I_p$ |
| $e_{max}$ | void ratio in loosest state |
| $e_{min}$ | void ratio in densest state |
| $I_D$ | density index = $(e_{max} - e) / (e_{max} - e_{min})$<br>(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 |
| $\sigma'_p$ | pre-consolidation pressure |
| OCR | over-consolidation ratio = $\sigma'_p/\sigma'_{vo}$ |

### (d) Shear Strength

| | |
|---|---|
| $\tau_p, \tau_r$ | peak and residual shear strength |
| $\phi'$ | effective angle of internal friction |
| $\delta$ | angle of interface friction |
| $\mu$ | 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 $\rho$. Unit weight symbol is $\gamma$ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

Golder Associates
London, Ontario

# RECORD OF BOREHOLE No 101

1 OF 1 **METRIC**

| | | | |
|---|---|---|---|
| PROJECT 08-1132-084-1 | | | |
| W.P. 131-98-00 | LOCATION N 4810292.7 ;E 225365.5 | ORIGINATED BY JB | |
| DIST | HWY 7/8 | BOREHOLE TYPE POWER AUGER | COMPILED BY LMK/SL |
| DATUM GEODETIC | DATE October 23, 2008 - October 24, 2008 | CHECKED BY | |

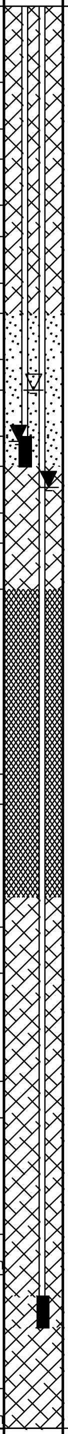
SOIL PROFILE

| ELEV<br>DEPTH | DESCRIPTION |
|---|---|
| 325.52 | GROUND SURFACE |
| 0.00 | TOPSOIL |
| 0.15 | FILL, sand & gravel, some silt<br>Dense<br>Brown |
| 324.30<br>1.22 | SAND, fine to medium, trace clay, trace silt, trace gravel<br>Dense<br>Brown |
| 320.95<br>4.57 | SILTY FINE SAND, trace clay<br>Compact<br>Brown |
| 318.21<br>7.31 | CLAYEY SILT, trace sand with silty fine sand layers<br>Very stiff<br>Brown |
| 316.84<br>8.68 | SILTY FINE SAND<br>Very dense<br>Grey |
| 315.16<br>10.36 | SAND AND GRAVEL, trace to some silt, trace clay<br>Very dense<br>Grey |
| 311.41<br>14.11 | END OF BOREHOLE<br><br>Groundwater encountered at about elev. 321.7m during drilling on October 23, 2008. |

SAMPLES

| NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|
| 1 | SS | 39 | 325 | | | | |
| 2 | SS | 38 | 324 | | | | |
| 3 | SS | 31 | 323 | 1 | 96 | 2 | 1 |
| 4 | SS | 32 | 322 | | | | |
| 5 | SS | 35 | | | | | |
| 6 | SS | 29 | 321 | | | | |
| 7 | SS | 18 | 320 | 0 | 71 | 24 | 5 |
| | | | 319 | | | | |
| 8 | SS | 17 | 318 | | | | |
| | | | 317 | | | | |
| 9 | SS | 66 | 316 | | | | |
| 10 | SS | 100 | 315 | | | | |
| | | | 314 | | | | |
| 11 | SS | 120/<br>230mm | 313 | 14 | 68 | 14 | 4 |
| 12 | SS | 115/<br>230mm | 312 | | | | |

GROUND WATER CONDITIONS



DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Water level in Shallow Piezometer at elev. 321.77m on October 24, 2008.
Water level in Deep Piezometer at elev. 321.31m on October 24, 2008.

Water level in Shallow Piezometer at elev. 321.71m on August 25, 2009.
Water level in Deep Piezometer at elev. 321.43m on August 25, 2009.

Water level in Shallow Piezometer at elev. 321.20m on June 3, 2010.
Water level in Deep Piezometer at elev. 320.74m on June 3, 2010.

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

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# RECORD OF BOREHOLE No 102

1 OF 1

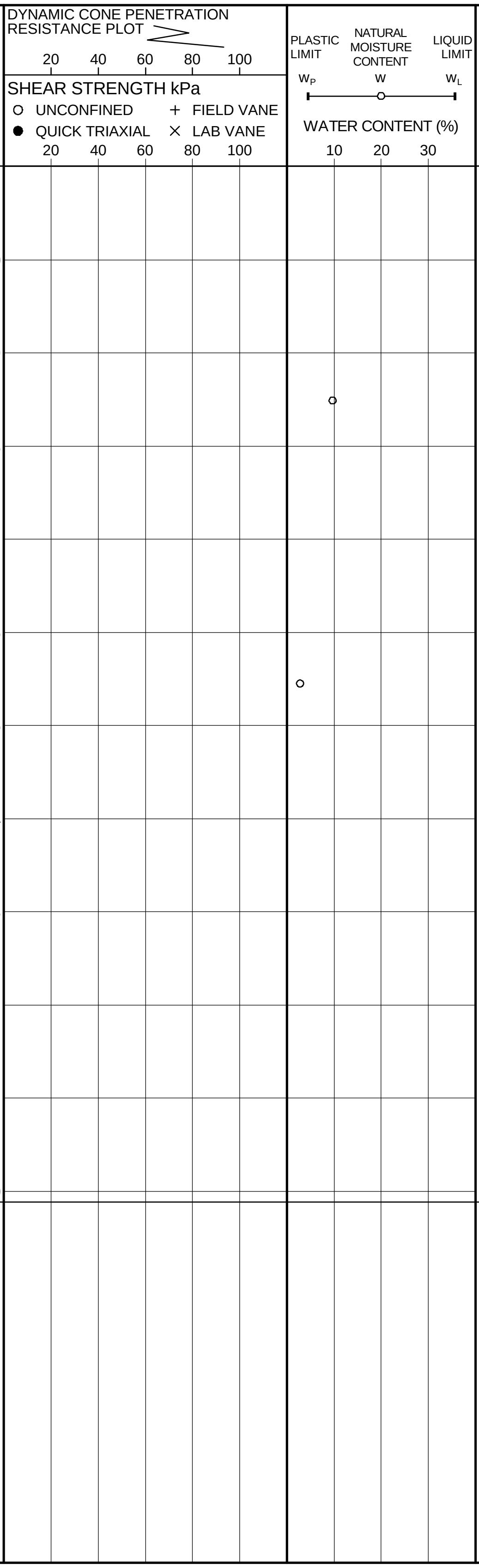
**METRIC**

PROJECT 08-1132-084-1
W.P. 131-98-00
LOCATION N 4810337.9 ;E 225388.1
ORIGINATED BY MA
DIST HWY 7/8
BOREHOLE TYPE POWER AUGER
COMPILED BY LMK/SL
DATUM GEODETIC
DATE October 24, 2008
CHECKED BY

| ELEV<br>DEPTH | DESCRIPTION (SOIL PROFILE) |
|---|---|
| 331.01 | PAVEMENT SURFACE |
| | ASPHALT |
| 0.11 | FILL, sand and gravel, crushed, trace silt<br>Brown |
| 330.58<br>0.43 | FILL, fine sand, some silt, trace gravel<br>Compact<br>Brown |
| 329.45<br>1.56 | FILL, clayey silt, some sand, trace gravel<br>Stiff<br>Brown |
| 1.86 | SILTY SAND, trace gravel, trace clay<br>Compact to dense<br>Brown |
| 326.59<br>4.42 | SAND, fine, trace silt<br>Dense<br>Brown |
| 325.83<br>5.18 | SAND, fine to medium, trace silt, some gravel, trace clay<br>Very dense<br>Brown |
| 324.61<br>6.40 | SAND, fine, some silt<br>Very dense<br>Brown |
| 319.88<br>11.13 | END OF BOREHOLE<br><br>Borehole dry upon completion of drilling on<br>October 24, 2008. |

| SAMPLES NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR | SA | SI | CL |
|---|---|---|---|---|---|---|
| 1 | SS | 19 | | | | |
| 2 | SS | 12 | | | | |
| 3 | SS | 18 | 3 | 56 | 31 | 10 |
| 4 | SS | 30 | | | | |
| 5 | SS | 42 | | | | |
| 6 | SS | 35 | | | | |
| 7 | SS | 69 | 18 | 71 | 8 | 3 |
| 8 | SS | 61 | | | | |
| 9 | SS | 52 | | | | |
| 10 | SS | 55 | | | | |
| 11 | SS | 54 | | | | |
| 12 | SS | 112 | | | | |

STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³



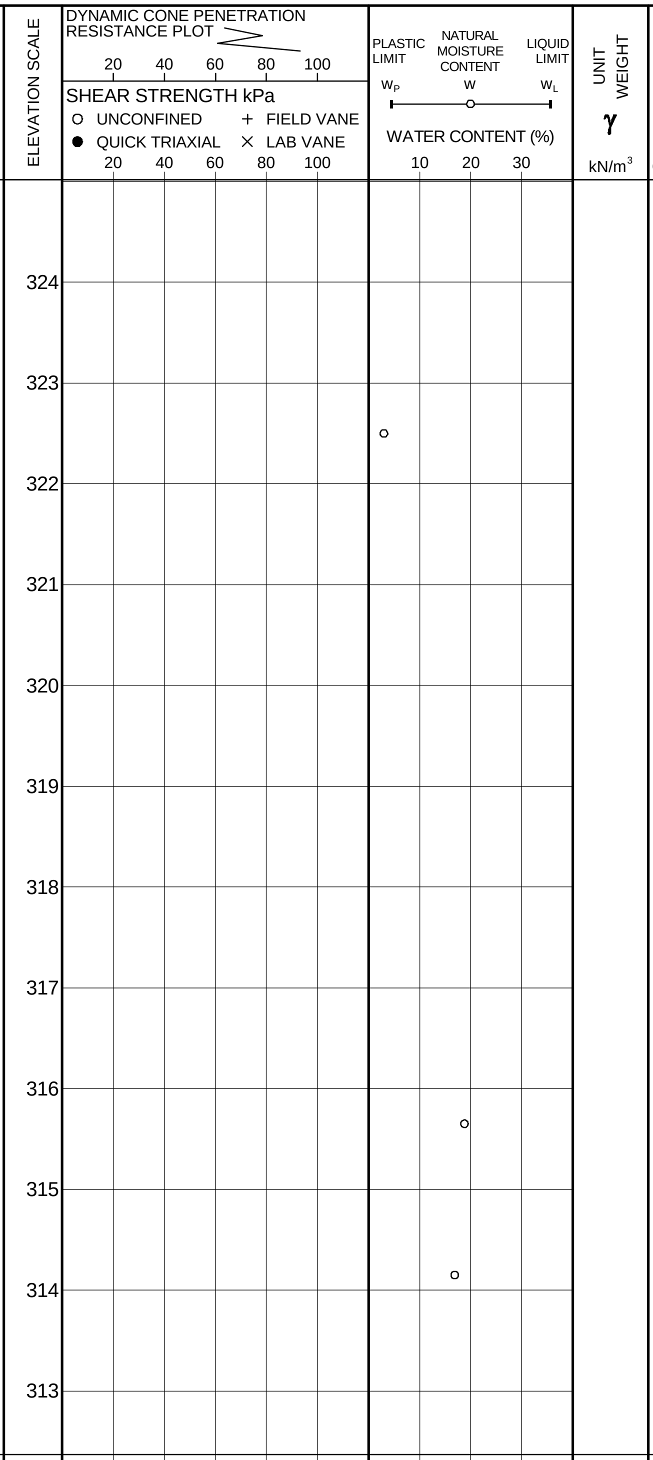
+³, ×³: Numbers refer to Sensitivity    ○³% STRAIN AT FAILURE

# RECORD OF BOREHOLE No 103

1 OF 1 **METRIC**

| | | | |
|---|---|---|---|
| PROJECT 08-1132-084-1 | | | |
| W.P. 131-98-00 | LOCATION N 4810285.9 ;E 225340.5 | ORIGINATED BY JB | |
| DIST | HWY 7/8 | BOREHOLE TYPE POWER AUGER | COMPILED BY LMK/SL |
| DATUM GEODETIC | DATE October 27, 2008 | CHECKED BY | |

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|
| 325.02 | GROUND SURFACE | | | | | | | | |
| 0.00 | TOPSOIL, rootlets | | | | | | | | |
| 324.41<br>0.61 | SILT, trace clay, trace gravel<br>Dense<br>Brown | 1 | SS | 39 | | | | | |
| 323.50<br>1.52 | SAND AND GRAVEL, trace silt<br>Very dense<br>Brown | 2 | SS | 58 | | | | | |
| 322.73<br>2.29 | SAND, fine to medium, trace silt, trace clay<br>Compact<br>Brown | 3 | SS | 27 | | 0 | 95 | 2 | 3 |
| 322.13<br>2.89 | SILTY FINE SAND, clayey silt seam at about elev. 318.9m<br>Compact to dense<br>Brown | 4 | SS | 39 | | | | | |
| | | | | | ∇ | | | | |
| | | 5 | SS | 15 | | | | | |
| | | 6 | SS | 10 | | | | | |
| | | 7 | SS | 10 | | | | | |
| | | 8 | SS | 13 | | | | | |
| 318.01<br>7.01 | SANDY SILT, trace clay, trace gravel<br>Compact to Very dense<br>Grey | 9 | SS | 27 | | | | | |
| | | 10 | SS | 112 | | 1 | 36 | 55 | 8 |
| 314.97<br>10.05 | SAND, fine to medium, some gravel, trace silt, trace clay<br>Very dense<br>Grey | 11 | SS | 116 | | 1 | 94 | 3 | 2 |
| | | 12 | SS | 104 | | | | | |
| 312.37<br>12.65 | END OF BOREHOLE<br><br>Groundwater encountered at about elev. 321.2m during drilling on October 27, 2008. | | | | | | | | |



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

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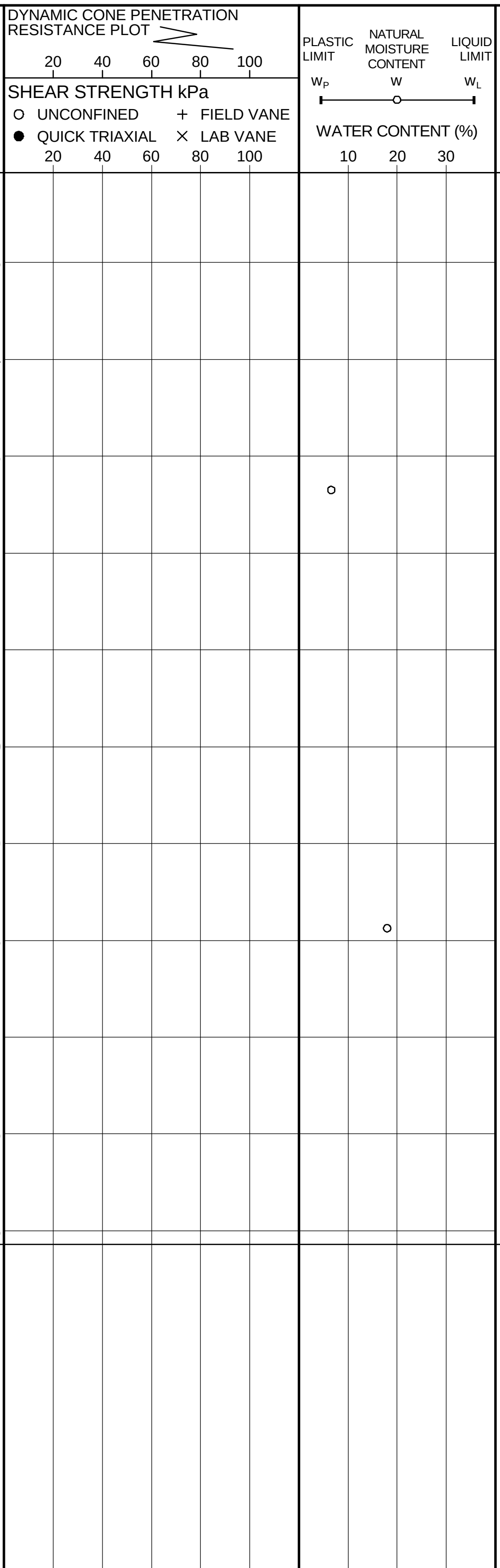
# RECORD OF BOREHOLE No 104

1 OF 1 **METRIC**

| | | | |
|---|---|---|---|
| PROJECT 08-1132-084-1 | | | |
| W.P. 131-98-00 | LOCATION N 4810331.7 ;E 225335.9 | ORIGINATED BY JB |
| DIST | HWY 7/8 | BOREHOLE TYPE POWER AUGER | COMPILED BY LMK/SL |
| DATUM GEODETIC | | DATE October 27, 2008 | CHECKED BY |

| ELEV<br>DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³ | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 325.93 | GROUND SURFACE | | | | | | | | | | |
| 0.00 | TOPSOIL | | | | | | | | | | |
| 0.30 | SILTY FINE SAND<br>Compact to dense<br>Brown | 1 | SS | 21 | | 325 | | | | | |
| | | 2 | SS | 40 | | 324 | | | | | |
| 323.64<br>2.29 | SILT, trace clay, trace sand<br>Very dense<br>Brown | 3 | SS | 79 | | 323 | | | | | |
| 322.88<br>3.05 | SILTY FINE SAND, trace clay<br>Very dense<br>Brown | 4 | SS | 106 | | | | 0 | 72 | 23 | 5 |
| | | 5 | SS | 120 | | 322 | | | | | |
| | | 6 | SS | 102 | | 321 | | | | | |
| 320.75<br>5.18 | SAND, fine, with coarse sand layers, trace gravel<br>Very dense<br>Brown | 7 | SS | 92 | ∇ | 320 | | | | | |
| 319.83<br>6.10 | SAND, fine, some silt, trace clay<br>Very dense<br>Brown | 8 | SS | 51 | | 319 | | | | | |
| | | 9 | SS | 107/<br>200mm | | 318 | | 0 | 86 | 11 | 3 |
| 317.40<br>8.53 | SILTY FINE SAND<br>Very dense<br>Grey | | | | | 317 | | | | | |
| | | 10 | SS | 124 | | 316 | | | | | |
| | | 11 | SS | 110/<br>250mm | | 315 | | | | | |
| 314.86<br>11.07 | END OF BOREHOLE<br><br>Groundwater encountered at about elev. 319.8m during drilling on October 27, 2008. | | | | | | | | | | |



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

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# RECORD OF BOREHOLE No 105

1 OF 2 **METRIC**

PROJECT 08-1132-084-1
W.P. 131-98-00
LOCATION N 4810336.3 ;E 225380.0
ORIGINATED BY MA/DB
DIST
HWY 7/8
BOREHOLE TYPE POWER AUGER / ROTARY DRILLING
COMPILED BY LMK/SL
DATUM GEODETIC
DATE October 30, 2008 - November 3, 2008
CHECKED BY

## SOIL PROFILE

| ELEV / DEPTH | DESCRIPTION |
|---|---|
| 331.14 | PAVEMENT SURFACE |
| 0.00 | ASPHALT |
| 0.12 | FILL, sand and gravel, crushed, trace silt<br>Brown |
| 330.62<br>0.52 | FILL, sand and gravel, trace silt<br>Compact<br>Brown |
| 329.77<br>1.37 | FILL, sandy silt<br>Loose to compact<br>Brown |
| 328.09<br>3.05 | FILL, silt, some clay, trace sand<br>Compact<br>Brown |
| 3.17 | SAND, fine to medium, trace to some silt, trace clay<br>Compact to Very dense<br>Brown |
| 320.78<br>10.36 | SILT, some clay, trace gravel, with clayey silt partings<br>Very dense<br>Brown |
| 320.35<br>10.79 | SILTY SAND AND GRAVEL<br>Very dense<br>Grey |
| 319.56<br>11.58 | SAND AND GRAVEL, trace silt, trace clay<br>Very dense<br>Grey |
| 318.04<br>13.10 | SAND, fine to medium, trace silt, trace gravel<br>Very dense |

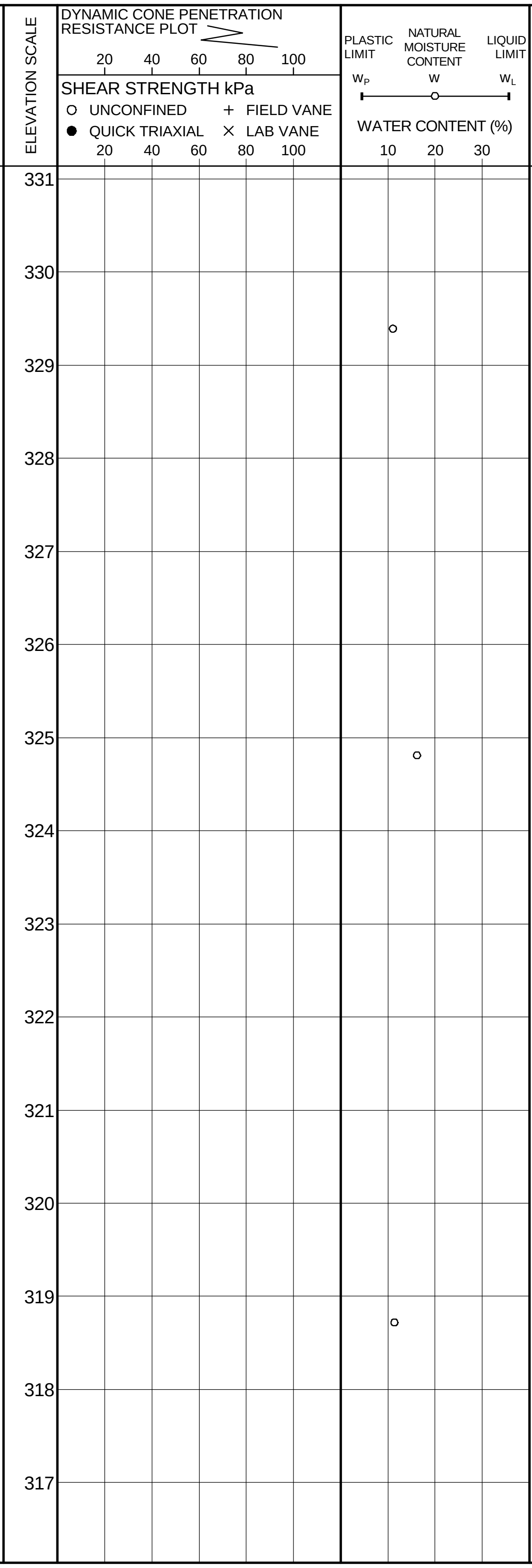
## SAMPLES

| NUMBER | TYPE | "N" VALUES | GR | SA | SI | CL |
|---|---|---|---|---|---|---|
| 1 | SS | 18 | | | | |
| 2 | SS | 16 | 0 | 47 | 47 | 6 |
| 3 | SS | 5 | | | | |
| 4 | SS | 17 | | | | |
| 5 | SS | 35 | | | | |
| 6 | SS | 55 | | | | |
| 7 | SS | 51 | | | | |
| 8 | SS | 50 | 0 | 80 | 16 | 4 |
| 9 | SS | 41 | | | | |
| 10 | SS | 85 | | | | |
| 11 | SS | 81/<br>150mm | | | | |
| 12 | SS | 67 | 25 | 67 | 5 | 3 |
| 13 | SS | 74 | | | | |

GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m³

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



Continued Next Page

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

O $^{3\%}$ STRAIN AT FAILURE

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London, Ontario

## RECORD OF BOREHOLE No 105

2 OF 2 **METRIC**

PROJECT 08-1132-084-1

W.P. 131-98-00

LOCATION N 4810336.3 ;E 225380.0

ORIGINATED BY MA/DB

DIST HWY 7/8

BOREHOLE TYPE POWER AUGER / ROTARY DRILLING

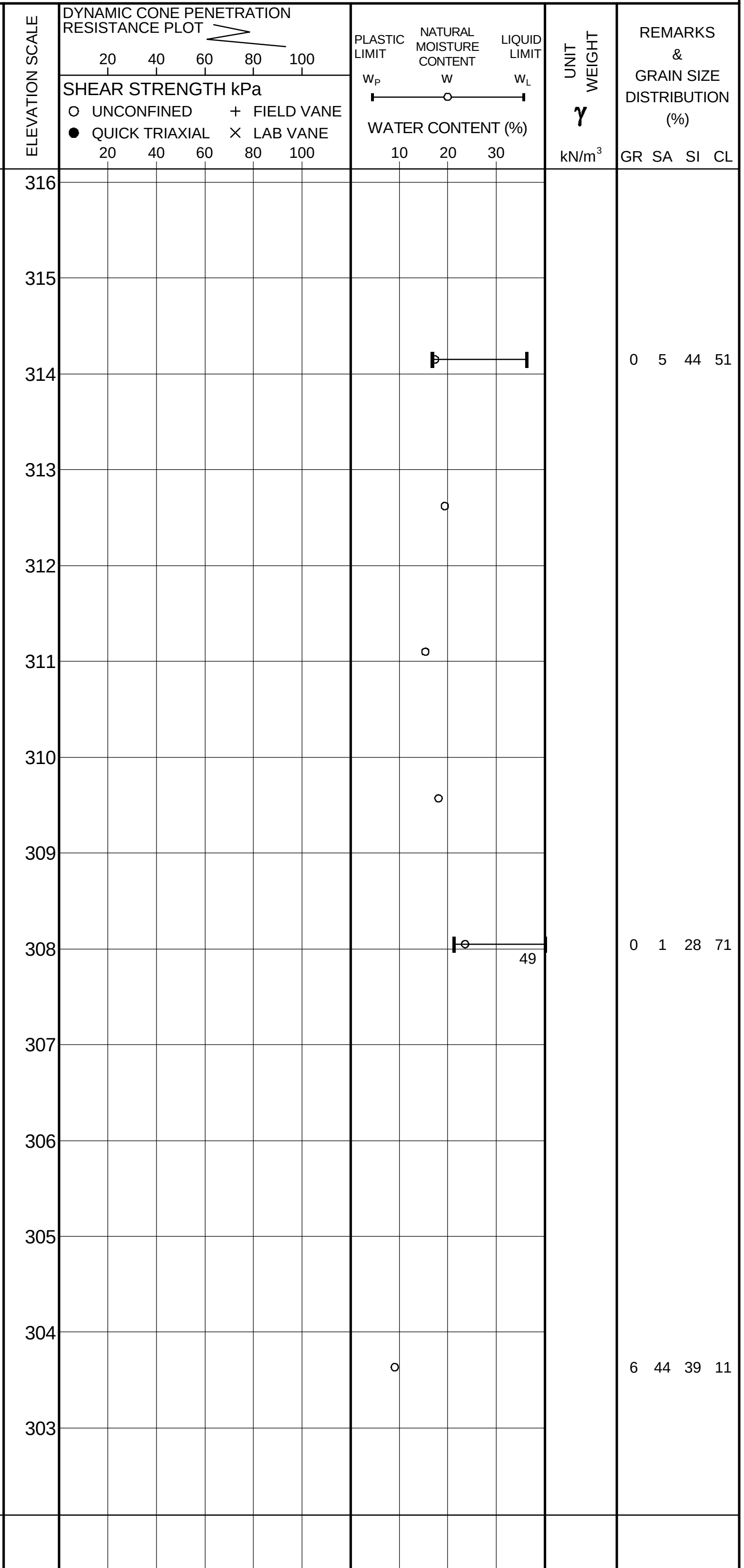
COMPILED BY LMK/SL

DATUM GEODETIC

DATE October 30, 2008 - November 3, 2008

CHECKED BY

| ELEV<br>DEPTH | SOIL PROFILE – DESCRIPTION | SAMPLES – NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|
|  | SAND, fine to medium, trace silt, trace gravel<br>Very dense | 14 | SS | 84 |  |  |  |  |  |
| 315.29<br>15.85 | SILTY CLAY TILL, trace sand, trace gravel<br>Hard<br>Grey | 15 | SS | 100 |  | 0 | 5 | 44 | 51 |
|  |  | 16 | SS | 88 |  |  |  |  |  |
|  |  | 17 | SS | 103 |  |  |  |  |  |
|  |  | 18 | SS | 90 |  |  |  |  |  |
|  |  | 19 | SS | 62 |  | 0 | 1 | 28 | 71 |
|  |  | 20 | SS | 78 |  |  |  |  |  |
| 305.99<br>25.15 | SANDY SILT TILL, some clay, trace to some gravel<br>Very dense<br>Grey | 21 | SS | 100/<br>75mm |  |  |  |  |  |
|  |  | 22 | SS | 100/<br>150mm |  | 6 | 44 | 39 | 11 |
| 302.09<br>29.05 | END OF BOREHOLE<br><br>Borehole dry upon completion of drilling on<br>November 3, 2008. | 23 | SS | 100/<br>75mm |  |  |  |  |  |



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

Golder Associates
London, Ontario

# RECORD OF BOREHOLE No 106

1 OF 1 **METRIC**

| | | | |
|---|---|---|---|
| PROJECT 08-1132-084-1 | | | |
| W.P. 131-98-00 | LOCATION N 4810322.4 ;E 225305.2 | ORIGINATED BY MA | |
| DIST | HWY 7/8 | BOREHOLE TYPE POWER AUGER | COMPILED BY LMK/SL |
| DATUM GEODETIC | DATE October 30, 2008 | CHECKED BY | |

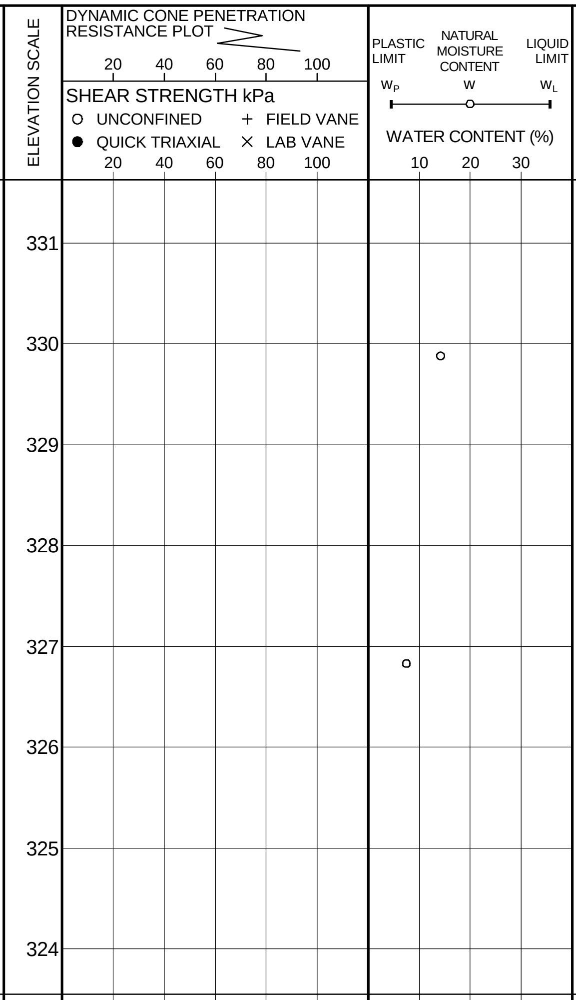
## SOIL PROFILE

| ELEV DEPTH | DESCRIPTION |
|---|---|
| 331.63 | PAVEMENT SURFACE |
| 0.00 | ASPHALT |
| 0.15 | FILL, sand and gravel, crushed, trace silt<br>Brown |
| 0.37 | FILL, sand and gravel, trace silt, with cobbles<br>Brown |
| 0.67 | FILL, sandy silt, trace gravel with clayey silt layers<br>Compact<br>Brown |
| 329.04<br>2.59 | FILL, sand, fine to medium, trace gravel<br>Compact<br>Brown |
| 328.73<br>2.90 | FILL, sandy silt, trace gravel, trace topsoil<br>Compact<br>Brown |
| 327.97<br>3.66 | SANDY SILT, with fine sand layers<br>Very dense<br>Brown |
| 323.55<br>8.08 | END OF BOREHOLE<br>Borehole dry upon completion of drilling on October 30, 2008. |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | UNIT WEIGHT γ kN/m³ | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|
| 1 | SS | 10 | | | | | | |
| 2 | SS | 22 | | | 3 | 34 | 50 | 13 |
| 3 | SS | 19 | | | | | | |
| 4 | SS | 28 | | | | | | |
| 5 | SS | 62 | | | | | | |
| 6 | SS | 61 | | | 0 | 28 | 66 | 6 |
| 7 | SS | 57 | | | | | | |
| 8 | SS | 61 | | | | | | |

REMARKS & GRAIN SIZE DISTRIBUTION (%)



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

Golder Associates
London, Ontario

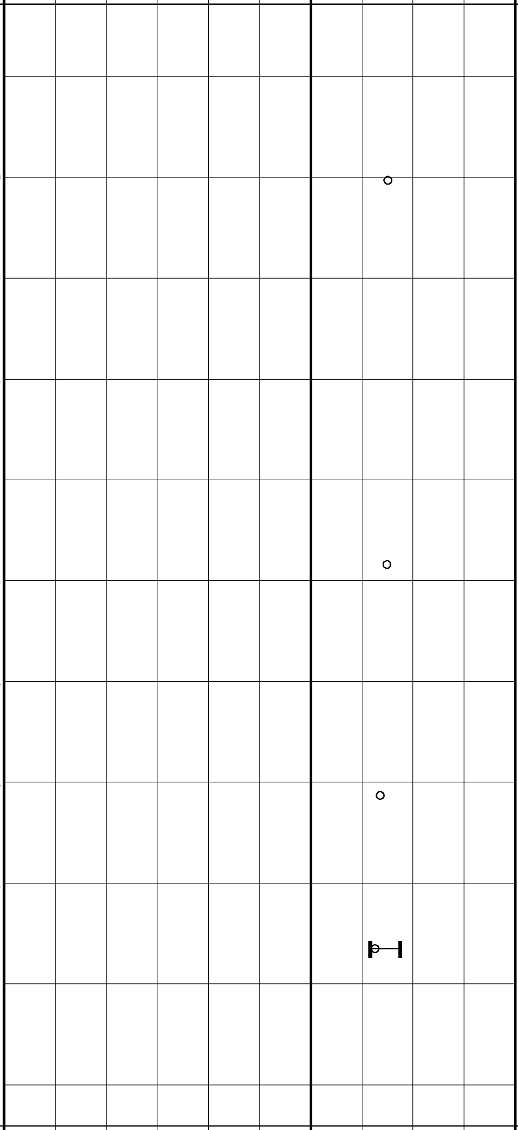
# RECORD OF BOREHOLE No 107

1 OF 1 **METRIC**

PROJECT 08-1132-084-1
W.P. 131-98-00
LOCATION N 4810280.5 ;E 225308.2
ORIGINATED BY MA/DB
DIST
HWY 7/8
BOREHOLE TYPE POWER AUGER
COMPILED BY LMK/SL
DATUM GEODETIC
DATE October 31, 2008
CHECKED BY

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT $W_P$ — NATURAL MOISTURE CONTENT W — LIQUID LIMIT $W_L$
WATER CONTENT (%): 10 20 30
UNIT WEIGHT γ kN/$m^3$

| ELEV DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|
| 331.72 | PAVEMENT SURFACE | | | | | | | | | |
| 0.00 | ASPHALT | | | | | | | | | |
| 0.15 | FILL, sand and gravel, crushed, trace silt<br>Brown | | | | | | | | | |
| 331.17<br>0.55 | FILL, silty sand, trace clay, trace gravel, clayey silt pockets<br>Loose to dense<br>Brown | | | | | 331 | | | | |
| | | 1 | SS | 4 | | | | | | |
| | | 2 | SS | 5 | | 330 | 4 | 49 | 40 | 7 |
| | | 3 | SS | 37 | | 329 | | | | |
| | | 4 | SS | 15 | | 328 | | | | |
| | | 5 | SS | 12 | | | | | | |
| 327.30<br>4.42 | SANDY SILT, trace clay<br>Compact to loose<br>Brown | | | | | 327 | | | | |
| | | 6 | SS | 23 | | | | | | |
| | | 7 | SS | 17 | | 326 | 0 | 44 | 49 | 7 |
| | | 8 | SS | 8 | | | | | | |
| 325.01<br>6.71 | SANDY SILT, trace gravel, trace clay, with clayey silt layers<br>Loose to compact<br>Brown | | | | | 325 | | | | |
| | | 9 | SS | 8 | ∇ | | | | | |
| | | 10 | SS | 21 | | 324 | 0 | 23 | 59 | 18 |
| | | | | | | 323 | | | | |
| | | 11 | SS | 7 | | 322 | 1 | 25 | 57 | 17 |
| 321.66<br>10.06 | SILT, some clay, trace sand, with clayey silt layers<br>Compact<br>Grey | | | | | 321 | | | | |
| | | 12 | SS | 21 | | | | | | |
| 320.59<br>11.13 | END OF BOREHOLE<br>Groundwater encountered at about elev. 324.2m during drilling on October 31, 2008. | | | | | | | | | |



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

LDN_MTO_06 08-1132-084-1.GPJ LDN_MTO.GDT 06/07/10

Golder Associates
London, Ontario

# RECORD OF BOREHOLE No 108

1 OF 2

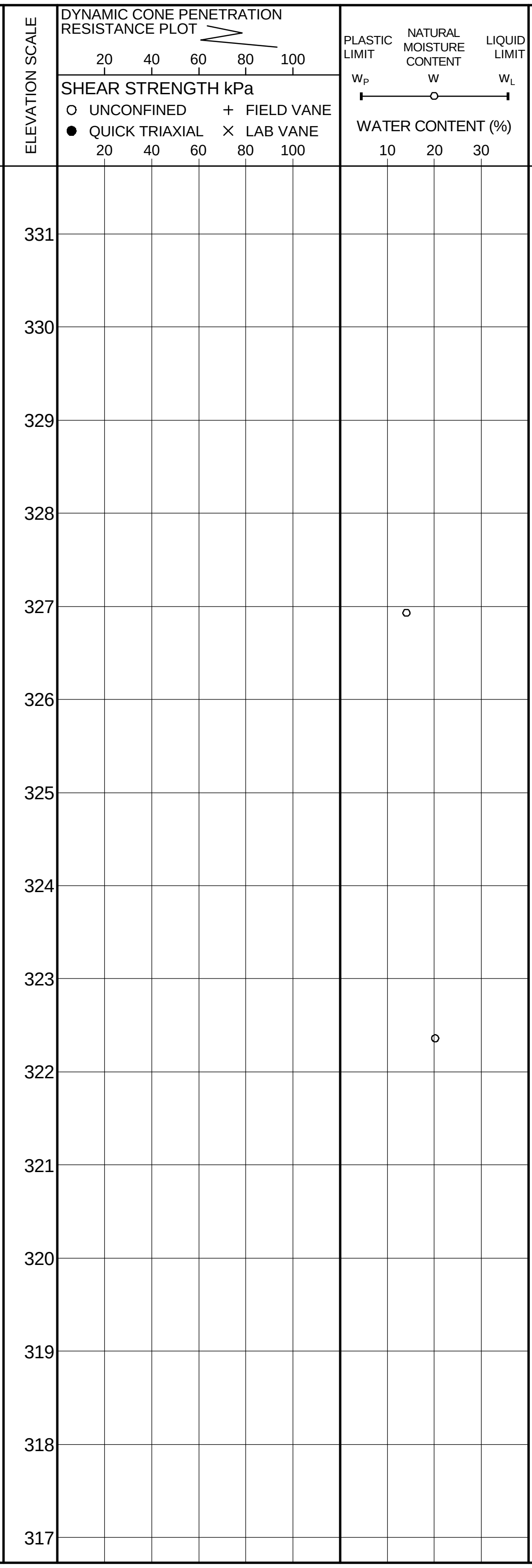
**METRIC**

PROJECT 08-1132-084-1
W.P. 131-98-00
LOCATION N 4810285.4 ;E 225324.5
ORIGINATED BY MA
DIST
HWY 7/8
BOREHOLE TYPE POWER AUGER / ROTARY DRILLING
COMPILED BY LMK/SL
DATUM GEODETIC
DATE November 4, 2008
CHECKED BY

| SOIL PROFILE | | | SAMPLES | | | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GR | SA | SI | CL |
| 331.73 | PAVEMENT SURFACE | | | | | | | | |
| 0.00 | ASPHALT | | | | | | | | |
| 0.15 | CONCRETE | | | | | | | | |
| 331.27<br>0.46 | FILL, sand and gravel, crushed, trace silt<br>Brown | | | | | | | | |
| 0.61 | FILL, sand, fine to medium, trace to some gravel<br>Dense<br>Brown | | 1 | SS | 48 | | | | |
| | | | 2 | SS | 31 | | | | |
| 329.87<br>1.86 | FILL, sandy silt, trace to some clay<br>Loose to compact<br>Brown | | 3 | SS | 8 | | | | |
| | | | 4 | SS | 11 | | | | |
| | | | 5 | SS | 8 | | | | |
| 327.31<br>4.42 | SANDY SILT<br>Compact<br>Brown | | 6 | SS | 11 | 0 | 38 | 51 | 11 |
| 326.55<br>5.18 | CLAYEY SILT, trace to some sand, trace gravel<br>Stiff<br>Brown | | 7 | SS | 14 | | | | |
| | | | 8 | SS | 9 | | | | |
| 324.57<br>7.16 | SILT, trace clay, trace sand, trace gravel, with clayey silt and sandy silt layers<br>Dense<br>Brown | | 9 | SS | 41 | | | | |
| 323.20<br>8.53 | SAND, fine to medium, trace silt<br>Compact<br>Brown | | 10 | SS | 19 | 0 | 92 | 4 | 4 |
| | | | 11 | SS | 20 | | | | |
| 320.70<br>11.03 | SILT, trace to some sand, trace clay<br>Compact<br>Brown | | 12 | SS | 24 | | | | |
| 318.62<br>13.11 | SILTY FINE SAND<br>Dense<br>Grey | | 13 | SS | 49 | | | | |
| 317.68<br>14.05 | SILTY SAND AND GRAVEL,<br>Dense Grey | | | | | | | | |
| 317.25<br>14.48 | | | | | | | | | |

GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m³



Continued Next Page

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

○ 3% STRAIN AT FAILURE

Golder Associates
London, Ontario

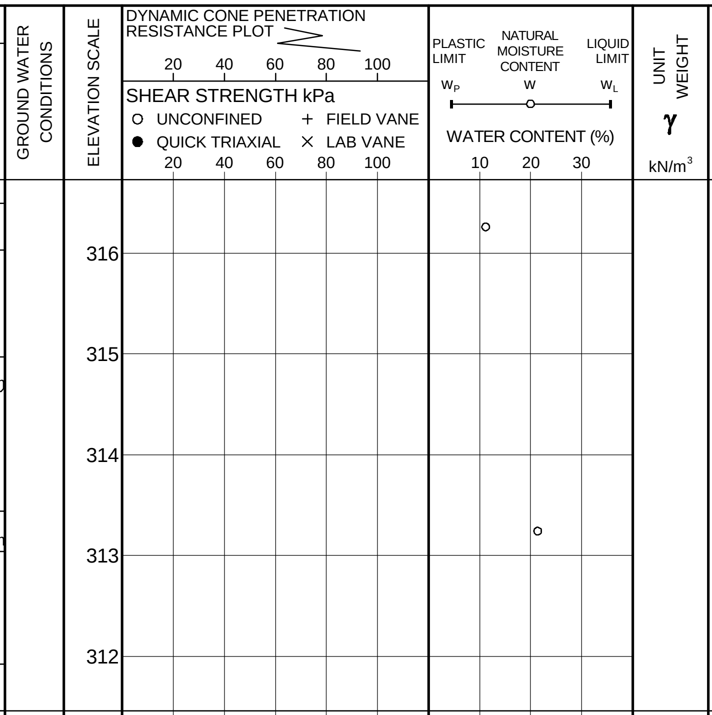
# RECORD OF BOREHOLE No 108

2 OF 2

**METRIC**

PROJECT 08-1132-084-1
W.P. 131-98-00
LOCATION N 4810285.4 ;E 225324.5
ORIGINATED BY MA
DIST
HWY 7/8
BOREHOLE TYPE POWER AUGER / ROTARY DRILLING
COMPILED BY LMK/SL
DATUM GEODETIC
DATE November 4, 2008
CHECKED BY

| SOIL PROFILE |  | SAMPLES |  |  | REMARKS & GRAIN SIZE DISTRIBUTION (%) |  |  |  |
|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GR | SA | SI | CL |
|  | SANDY SILT, trace to some clay, trace gravel<br>Very dense<br>Grey | 14 | SS | 92 | 0 | 42 | 47 | 11 |
|  |  | 15 | SS | 75/ 150mm |  |  |  |  |
|  |  | 16 | SS | 107/ 250mm | 0 | 26 | 70 | 4 |
| 312.53<br>19.20 | SILTY FINE SAND<br>Very dense<br>Grey | 17 | SS | 76 |  |  |  |  |
| 311.46<br>20.27 | END OF BOREHOLE<br>Borehole dry upon completion of drilling on<br>November 4, 2008. |  |  |  |  |  |  |  |



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

Golder Associates
London, Ontario

# RECORD OF BOREHOLE No 109

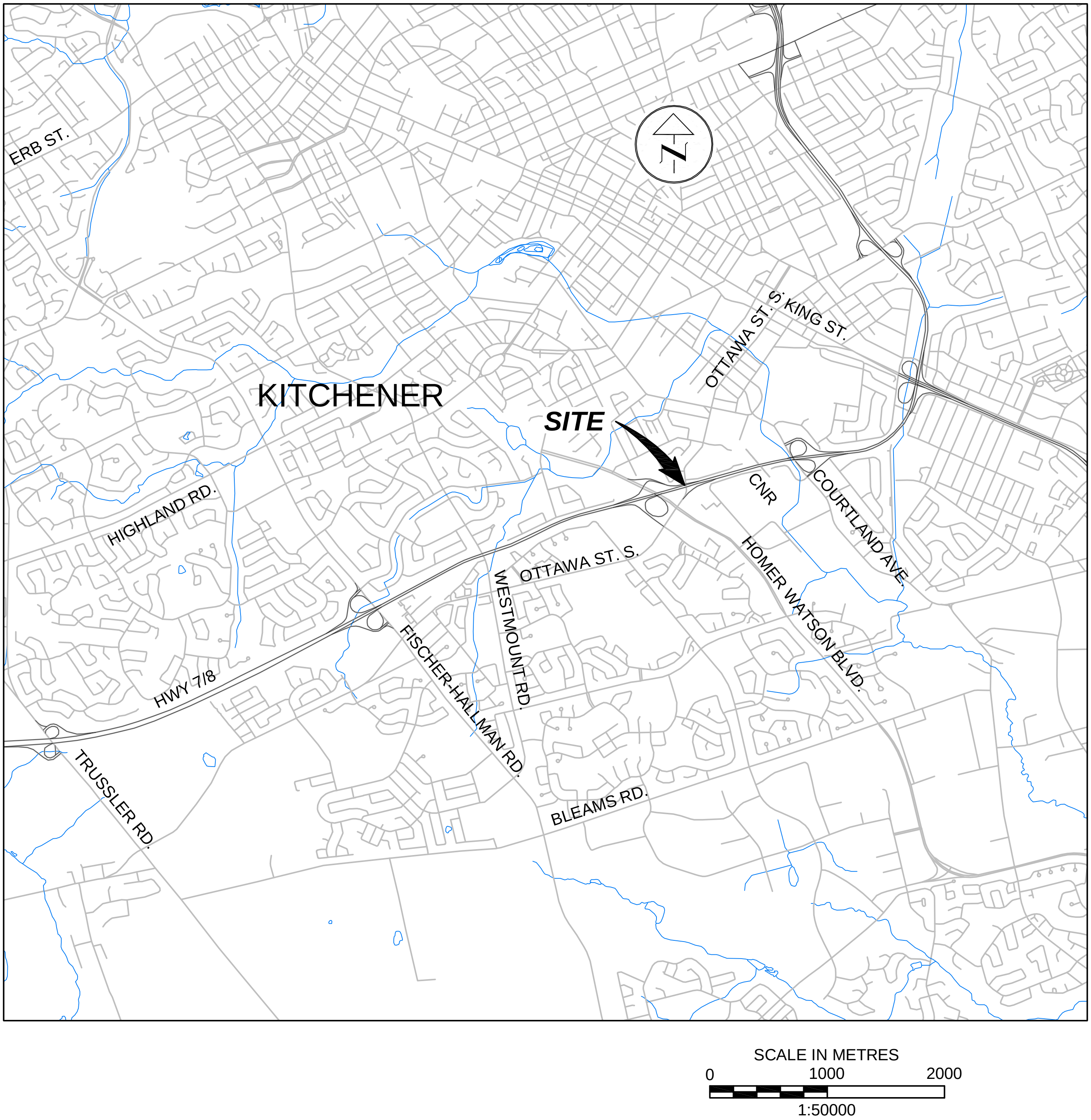
1 OF 1 **METRIC**

PROJECT 08-1132-084-1
W.P. 131-98-00
LOCATION N 4810306.2 ;E 225392.2
ORIGINATED BY MA
DIST
HWY 7/8
BOREHOLE TYPE POWER AUGER
COMPILED BY LMK/SL
DATUM GEODETIC
DATE November 4, 2008
CHECKED BY

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

| ELEV<br>DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | WATER CONTENT (%) | UNIT WEIGHT γ kN/$m^3$ | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 331.13 | PAVEMENT SURFACE | | | | | | | | | | | | |
| 0.00 | ASPHALT | ■ | | | | | | | | | | | |
| 0.12 | FILL, sand and gravel, crushed, trace silt<br>Brown | ▨ | | | | | 331 | | | | | | |
| 0.40 | FILL, silty fine sand with silt layers<br>Compact to very loose<br>Brown | ▨ | 1 | SS | 16 | | 330 | | | | | | |
| | | | 2 | SS | 3 | | | ○ | | 0 | 69 | 26 | 5 |
| | | | 3 | SS | 3 | | 329 | | | | | | |
| 328.23<br>2.90 | SILTY FINE SAND with fine to medium sand layers<br>Compact to dense<br>Brown | | 4 | SS | 21 | | 328 | ○ | | 0 | 66 | 29 | 5 |
| | | | 5 | SS | 30 | | 327 | | | | | | |
| | | | 6 | SS | 32 | | | | | | | | |
| 325.95<br>5.18 | SILT, some sand, trace clay<br>Dense<br>Brown | | 7 | SS | 36 | | 326 | | | | | | |
| 325.19<br>5.94 | SILTY FINE SAND<br>Dense<br>Brown | | 8 | SS | 35 | | 325 | | | | | | |
| 324.58<br>6.55 | END OF BOREHOLE<br><br>Borehole dry during drilling on November 4, 2008. | | | | | | | | | | | | |

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



## REFERENCE

DRAWING BASED ON CANMAP STREETFILES V2005.4.

## NOTE

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

PROJECT

**OTTAWA STREET OVERPASS (SITE 33-226)**
**WIDENING OF HIGHWAY 7/8**
**GWP 131-98-00**

TITLE

**KEY PLAN**

Golder Associates
LONDON, ONTARIO

| | | | |
|---|---|---|---|
| PROJECT No. | | 08-1132-084-1 | FILE No. 0811320841-F01001 |
| | | | SCALE AS SHOWN REV. |
| CADD | PH/WF/DH | July 06/10 | **FIGURE 1** |
| CHECK | | | |

Drawing file: 0811320841-F01001.DWG    Jul 06, 2010 - 10:42am

MINISTRY OF TRANSPORTATION, ONTARIO

CONT No.
WP No. 131–98–00

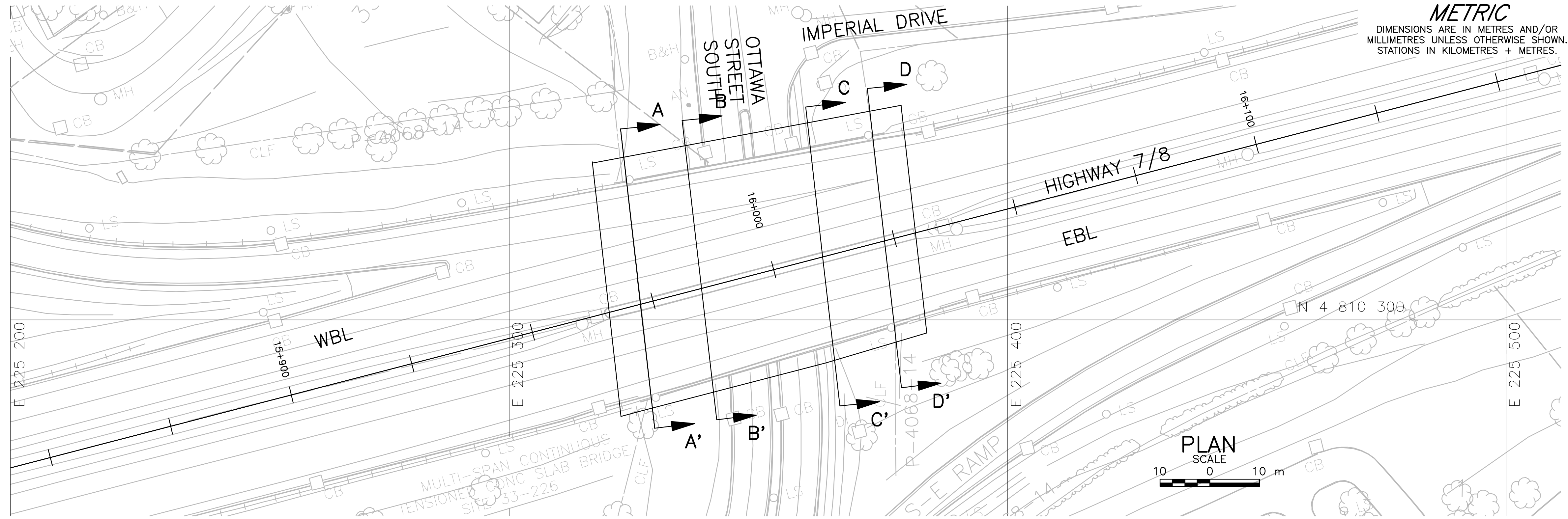
SHEET

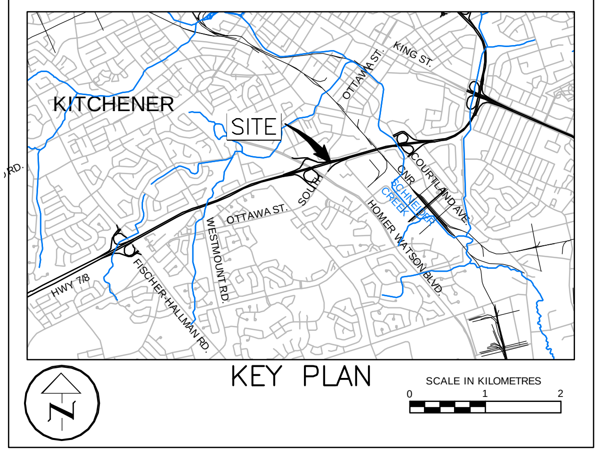
# OTTAWA STREET SOUTH OVERPASS

WIDENING OF HIGHWAY 7/8
BORDEHOLE LOCATIONS & SOIL STRATA

**Golder Associates Ltd.**
LONDON, ONTARIO, CANADA

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





## 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)
- ▮ Seal
- ▯ Standpipe
- *DRY* Borehole dry during drilling
- ▽ WL upon completion of drilling
- ▼ WL measured in standpipe (June 3, 2010)

| No. | ELEVATION | CO–ORDINATES (MTM ZONE 10) | |
|---|---|---|---|
| | | NORTHING | EASTING |
| 101 | 325.52 | 4 810 292.7 | 225 365.5 |
| 102 | 331.01 | 4 810 337.9 | 225 388.1 |
| 103 | 325.02 | 4 810 285.9 | 225 340.5 |
| 104 | 325.93 | 4 810 331.7 | 225 335.9 |
| 105 | 331.14 | 4 810 336.3 | 225 380.0 |
| 106 | 331.63 | 4 810 322.4 | 225 305.2 |
| 107 | 331.72 | 4 810 280.5 | 225 308.2 |
| 108 | 331.73 | 4 810 285.4 | 225 324.5 |
| 109 | 331.13 | 4 810 306.2 | 225 392.2 |
| By Others (Geocres #40P08–051) | | | |
| 1 | 328.48 | 4 810 321.9 | 225 322.2 |
| 2 | 329.03 | 4 810 287.6 | 225 337.7 |
| 3 | 329.70 | 4 810 329.9 | 225 361.5 |
| 4 | 332.57 | 4 810 295.5 | 225 374.2 |

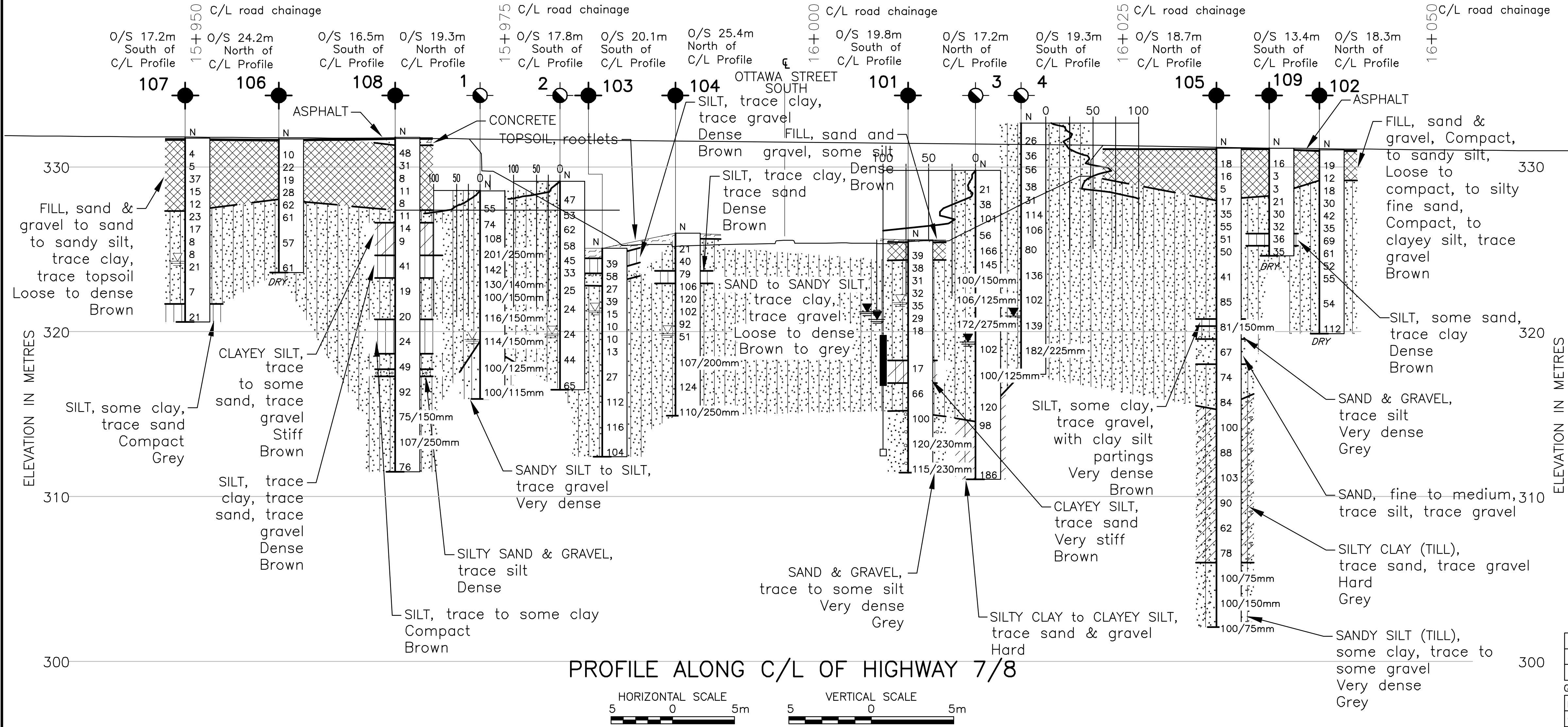
## NOTES

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

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

## REFERENCE

Base plans provided in digital format by Dillon Consulting.



PROFILE ALONG C/L OF HIGHWAY 7/8

HORIZONTAL SCALE
5 0 5m

VERTICAL SCALE
5 0 5m

| NO. | DATE | BY | REVISION |
|---|---|---|---|
| | | | |
| | | | |

Geocres No. 40P8–173

| | | | |
|---|---|---|---|
| HWY. 7/8 | | PROJECT NO. 08–1132–084–1 | DIST. |
| SUBM'D. DUP | CHKD. | DATE: July 06/10 | SITE: 33–226 |
| DRAWN: DH/LK/AG | CHKD. | APPD. | DWG. 1 |

PLOT DATE: July 16, 2010
FILENAME: N:\active\2008\1132 – Geotechnical\1132–000–0\08–1132–084–1 DILLON – GWP 131–98–00 FDNS – HWY 7–8\Drafting\AutoCAD files\0811320841–D01001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

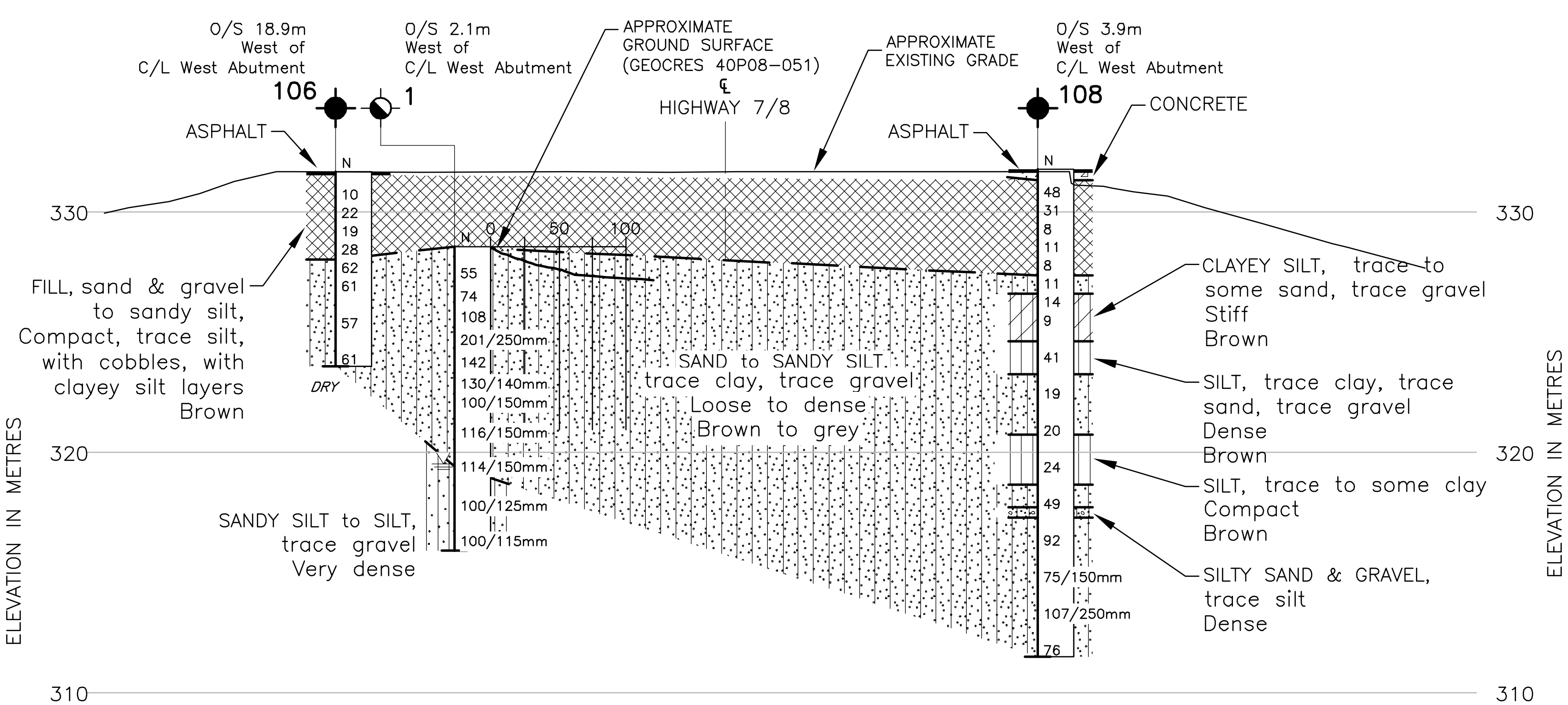
*METRIC*
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 131–98–00

OTTAWA STREET SOUTH OVERPASS
WIDENING OF HIGHWAY 7/8
SOIL STRATA

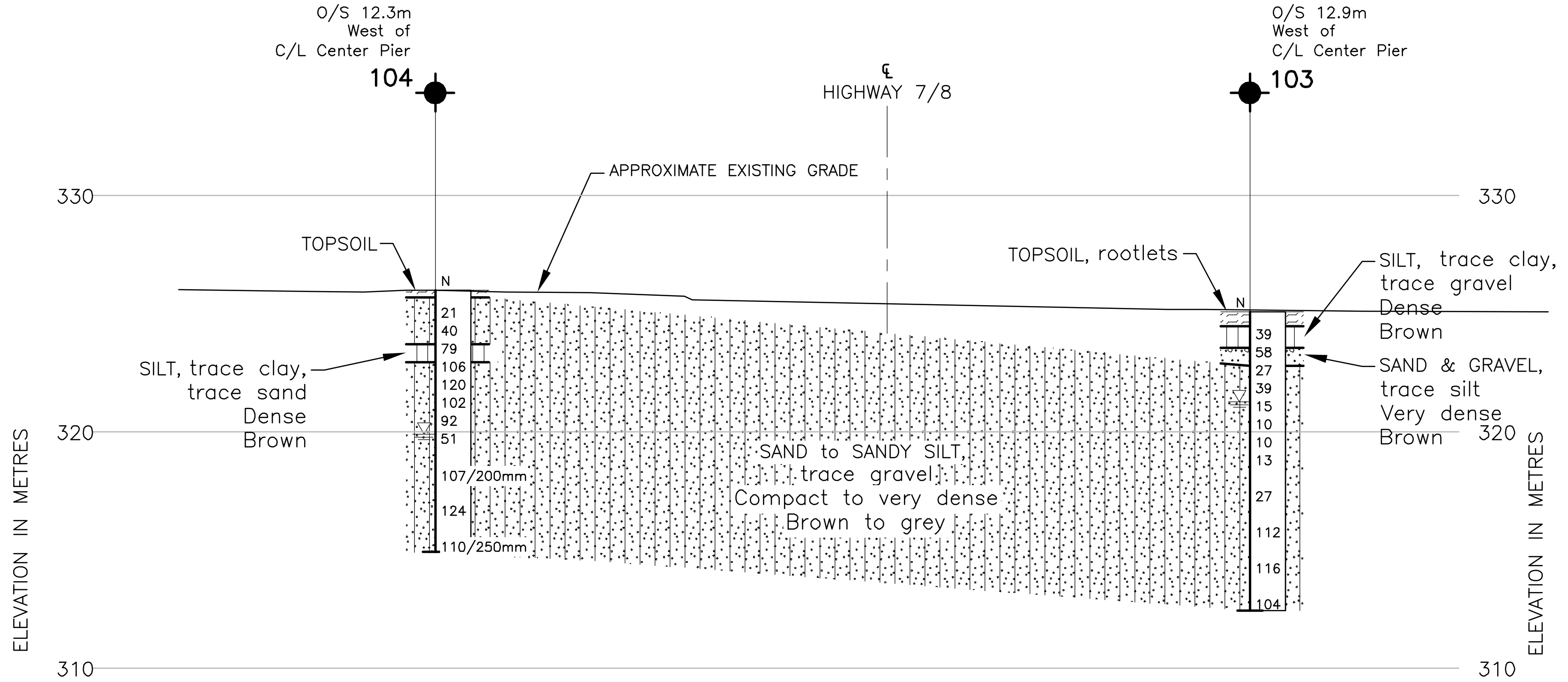
SHEET

**Golder Associates Ltd.**
LONDON, ONTARIO, CANADA



## SECTION A–A' ALONG WEST ABUTMENT

HORIZONTAL 5 0 5m
VERTICAL SCALE 5 0 5m



## SECTION B–B' ALONG WEST PIER

HORIZONTAL 5 0 5m
VERTICAL SCALE 5 0 5m

## 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 measured in standpipe (June 3, 2010) |

| No. | ELEVATION | CO–ORDINATES (MTM ZONE 10) | |
|---|---|---|---|
| | | NORTHING | EASTING |
| 103 | 325.02 | 4 810 285.9 | 225 340.5 |
| 104 | 325.93 | 4 810 331.7 | 225 335.9 |
| 106 | 331.63 | 4 810 322.4 | 225 305.2 |
| 108 | 331.73 | 4 810 285.4 | 225 324.5 |
| By Others (Geocres #40P08–051) | | | |
| 1 | 328.48 | 4 810 321.9 | 225 322.2 |

## NOTES

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

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

## REFERENCE

Base plans provided in digital format by Dillon Consulting.

| NO. | DATE | BY | REVISION |
|---|---|---|---|
| | | | |
| | | | |

Geocres No. 40P8–173

| | | | |
|---|---|---|---|
| HWY. 7/8 | | PROJECT NO. 08–1132–084–1 | DIST. |
| SUBM'D. DUP | CHKD. | DATE: July 06/10 | SITE: 33–226 |
| DRAWN: DH/LK/AG | CHKD. | APPD. | DWG. 2 |

PLOT DATE: July 16, 2010
FILENAME: N:\active\2008\1132 – Geotechnical\1132–000–0\08–1132–084–1 DILLON – GWP 131–98–00 FDNS – HWY 7–8\Drafting\AutoCAD files\0811320841–D01001.dwg

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

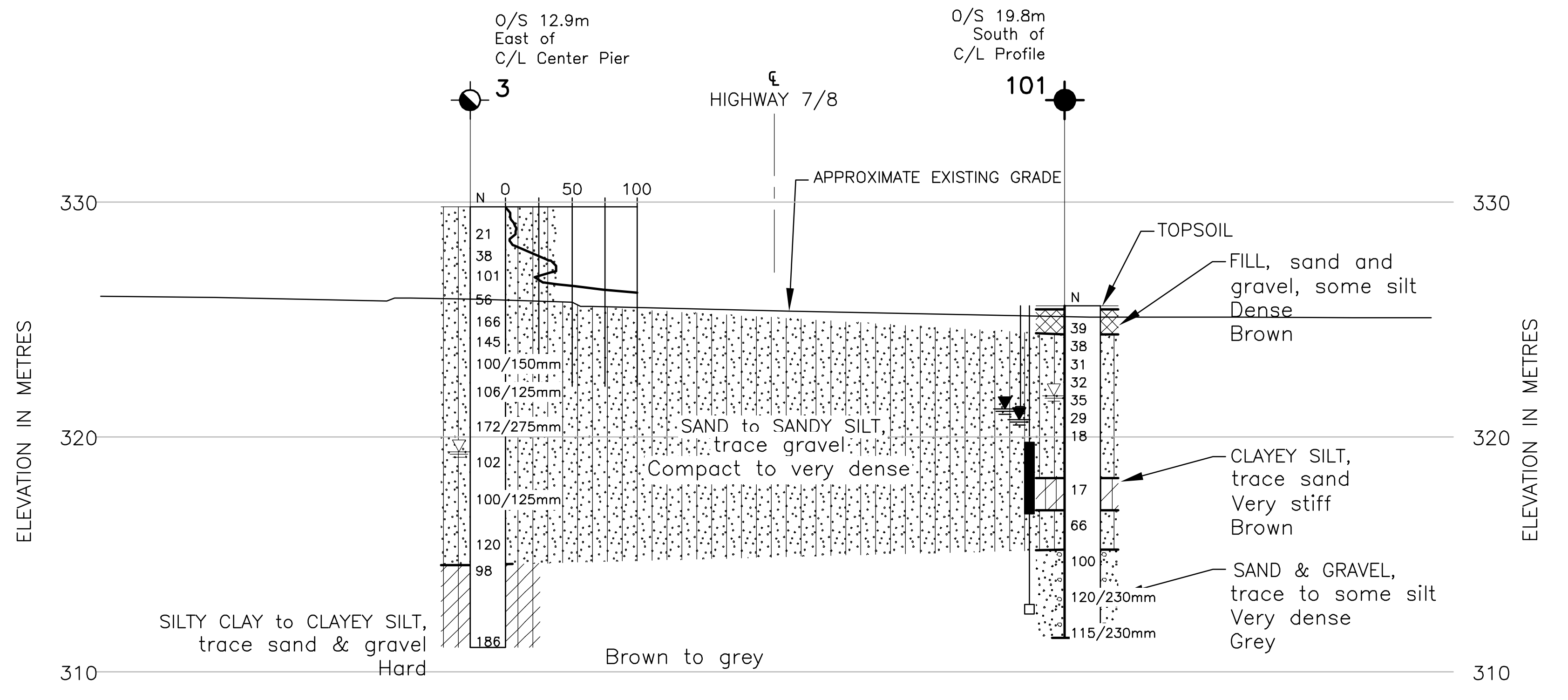
CONT No.
WP No. 131–98–00

SHEET

## OTTAWA STREET SOUTH OVERPASS

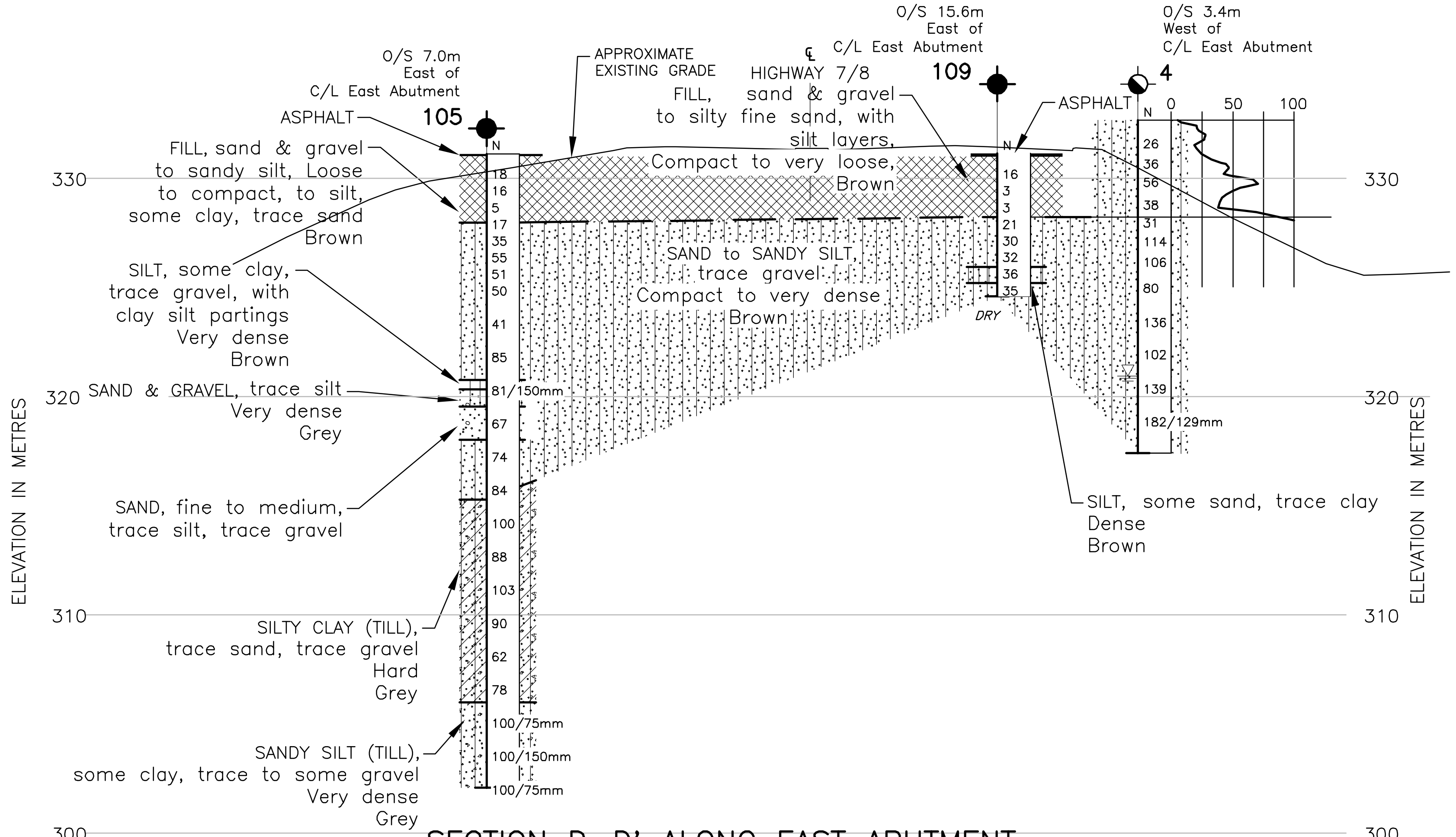
WIDENING OF HIGHWAY 7/8
SOIL STRATA

**Golder Associates Ltd.**
LONDON, ONTARIO, CANADA



## SECTION C–C' ALONG EAST PIER

HORIZONTAL 5 0 5m    VERTICAL SCALE 5 0 5m



## SECTION D–D' ALONG EAST ABUTMENT

HORIZONTAL 5 0 5m    VERTICAL SCALE 5 0 5m

### LEGEND

| | |
|---|---|
| ● | Borehole – Current Investigation |
| ◐ | Borehole (By Others) (Geocres #40P08–051) |
| N | Standard Penetration Test Value |
| 16 | Blows/0.3m unless otherwise stated<br>(Std. Pen. Test, 475 j/blow) |
| ▮ | Seal |
| ▯ | Standpipe |
| DRY | Borehole dry during drilling |
| ▽ | WL upon completion of drilling |
| ▼ | WL measured in standpipe (June 3, 2010) |

| No. | ELEVATION | CO–ORDINATES (MTM ZONE 10) | |
|---|---|---|---|
| | | NORTHING | EASTING |
| 101 | 325.52 | 4 810 292.7 | 225 365.5 |
| 105 | 331.14 | 4 810 322.4 | 225 305.2 |
| 109 | 331.13 | 4 810 306.2 | 225 392.2 |
| By Others (Geocres #40P08–051) | | | |
| 3 | 329.70 | 4 810 329.9 | 225 361.5 |
| 4 | 332.57 | 4 810 295.5 | 225 374.2 |

### NOTES

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

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

### REFERENCE

Base plans provided in digital format by Dillon Consulting.

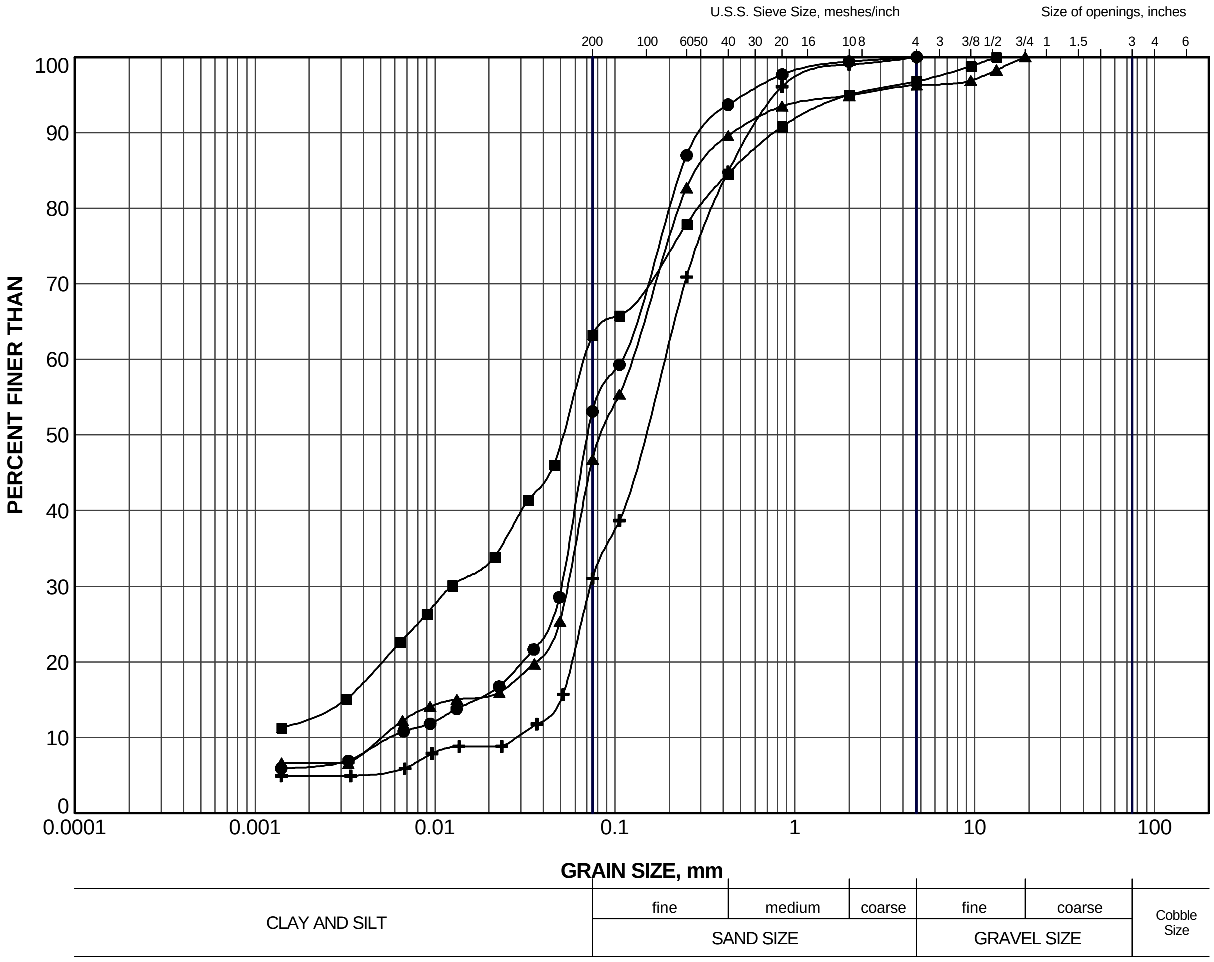
| NO. | DATE | BY | REVISION |
|---|---|---|---|
| | | | |

Geocres No. 40P8–173

| | | | |
|---|---|---|---|
| HWY. 7/8 | PROJECT NO. 08–1132–084–1 | | DIST. |
| SUBM'D. DUP | CHKD. | DATE: July 06/10 | SITE: 33–226 |
| DRAWN: DH/LK/AG | CHKD. | APPD. | DWG. 3 |

# APPENDIX A

## Laboratory Test Data



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|---|---|---|---|
| ● | 105 | 2 | 329.4 |
| ■ | 106 | 2 | 329.9 |
| ▲ | 107 | 2 | 330.0 |
| ✚ | 109 | 2 | 329.4 |

PROJECT
OTTAWA STREET SOUTH OVERPASS (Site 33-226)
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

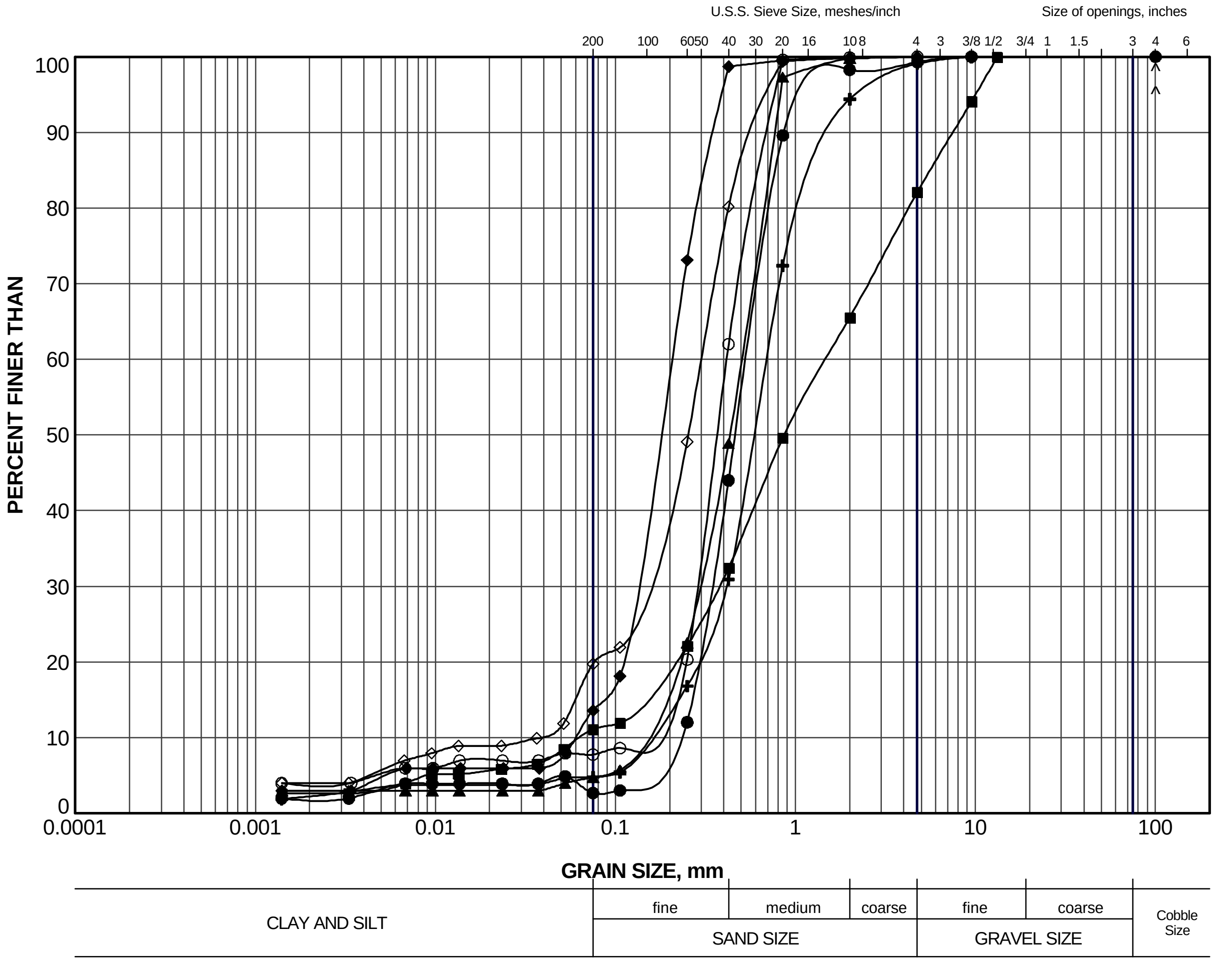
TITLE

# GRAIN SIZE DISTRIBUTION
## FILL

Golder Associates
LONDON, ONTARIO

| | | | | |
|---|---|---|---|---|
| PROJECT No. | 08-1132-084-1 | | FILE No. | 0811320841-R010A1 |
| | | | SCALE N/A | REV. |
| DRAWN | LMK | July 06/10 | **FIGURE A-1** | |
| CHECK | | | | |

LDN_MTO_NEW GLDR_LDN.GDT



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|---|---|---|---|
| ● | 101 | 3 | 323.0 |
| ■ | 102 | 7 | 325.5 |
| ▲ | 103 | 3 | 322.5 |
| ✚ | 103 | 11 | 314.2 |
| ◆ | 104 | 9 | 318.1 |
| ◇ | 105 | 8 | 324.8 |
| ○ | 108 | 10 | 322.4 |

PROJECT
OTTAWA STREET SOUTH OVERPASS (Site 33-226)
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE

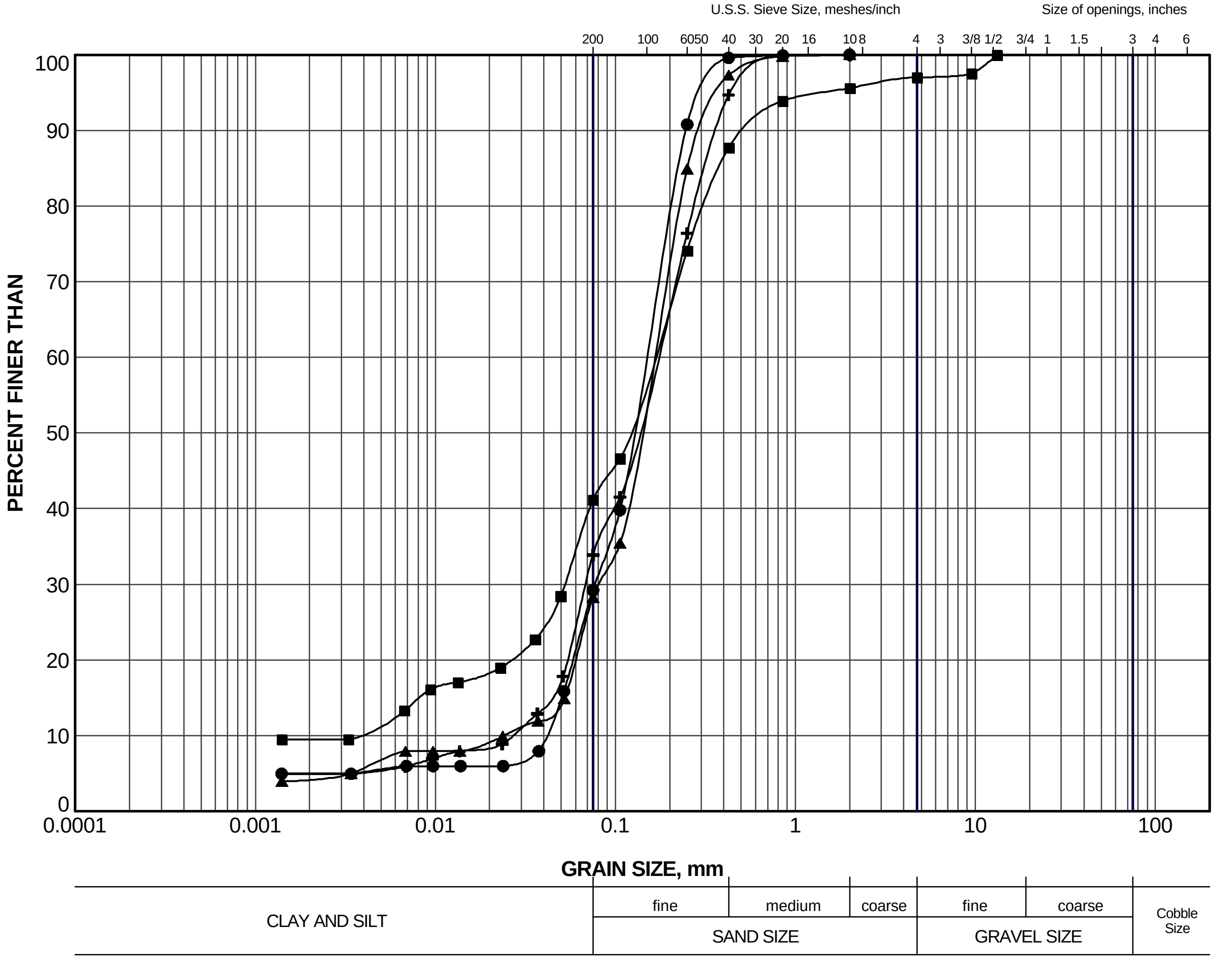
# GRAIN SIZE DISTRIBUTION

## SAND

Golder Associates
LONDON, ONTARIO

| | | | | |
|---|---|---|---|---|
| PROJECT No. | 08-1132-084-1 | | FILE No. | 0811320841-R010A2 |
| | | | SCALE N/A | REV. |
| DRAWN | DCH | July 06/10 | **FIGURE A-2** | |
| CHECK | | | | |

LDN_MTO_NEW GLDR_LDN.GDT



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|---|---|---|---|
| ● | 101 | 7 | 320.0 |
| ■ | 102 | 3 | 328.5 |
| ▲ | 104 | 4 | 322.7 |
| ✚ | 109 | 4 | 327.9 |

PROJECT
OTTAWA STREET SOUTH OVERPASS (Site 33-226)
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE

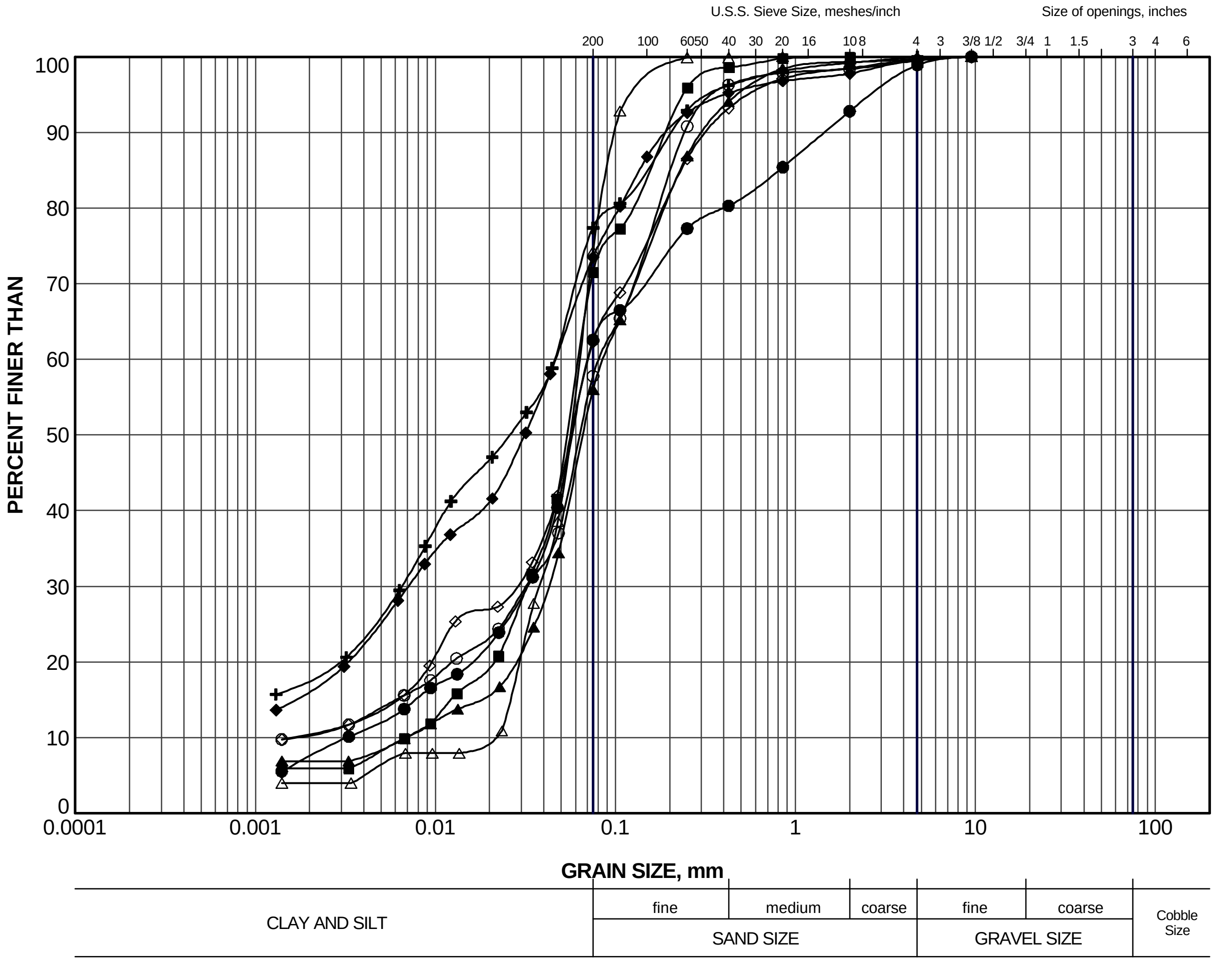
# GRAIN SIZE DISTRIBUTION

## SILTY FINE SAND

Golder Associates
LONDON, ONTARIO

| | | | |
|---|---|---|---|
| PROJECT No. | 08-1132-084-1 | FILE No. | 0811320841-R010A3 |
| | | SCALE N/A | REV. |
| DRAWN LMK | July 06/10 | **FIGURE A-3** | |
| CHECK | | | |

LDN_MTO_NEW GLDR_LDN.GDT



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|---|---|---|---|
| ● | 103 | 10 | 315.7 |
| ■ | 106 | 6 | 326.8 |
| ▲ | 107 | 7 | 326.2 |
| ✚ | 107 | 10 | 323.9 |
| ◆ | 107 | 11 | 322.4 |
| ◇ | 108 | 6 | 326.9 |
| ○ | 108 | 14 | 316.3 |
| △ | 108 | 16 | 313.2 |

PROJECT
OTTAWA STREET SOUTH OVERPASS (Site 33-226)
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE

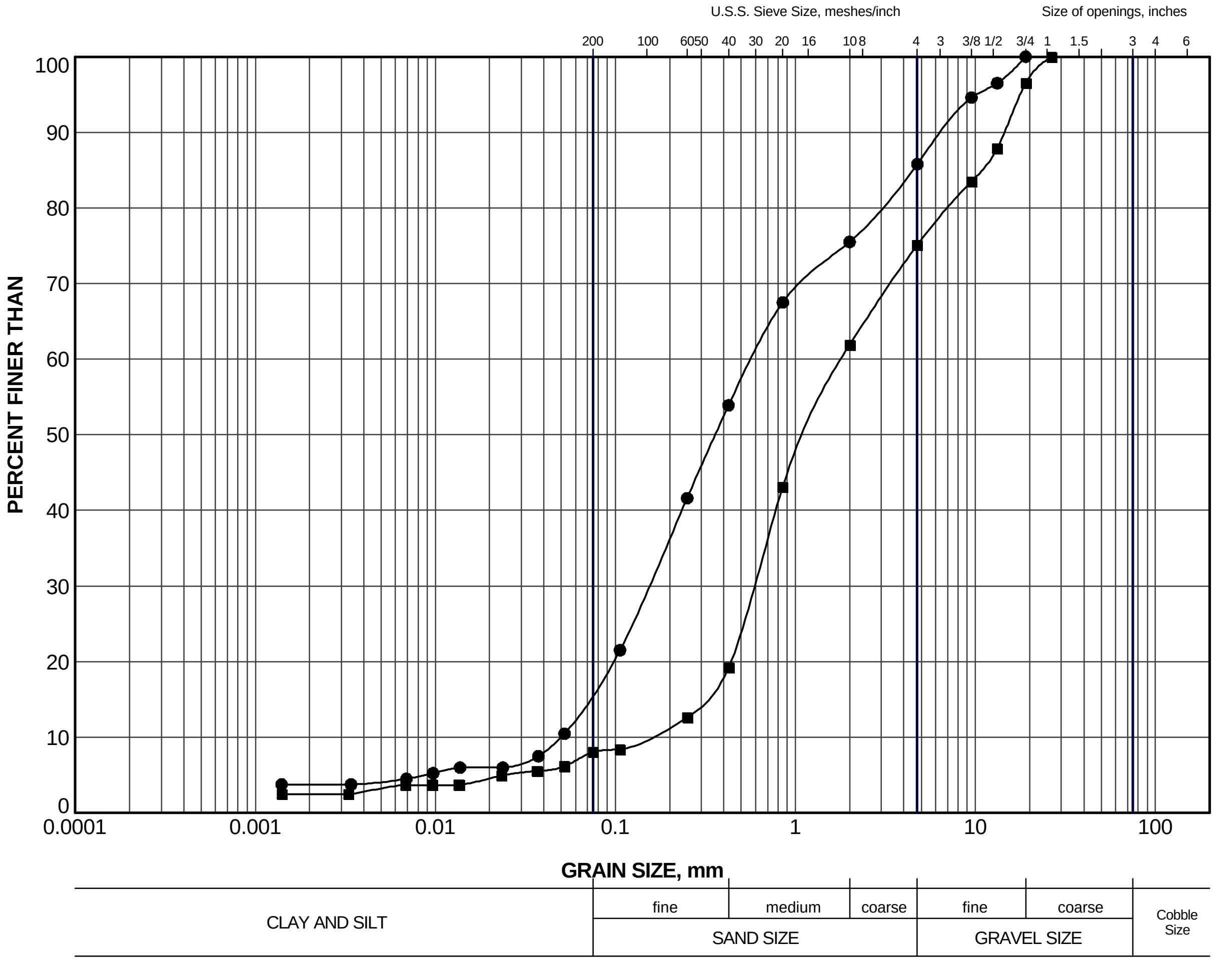
# GRAIN SIZE DISTRIBUTION

## SANDY SILT

Golder Associates
LONDON, ONTARIO

| | | | | |
|---|---|---|---|---|
| PROJECT No. | | 08-1132-084-1 | FILE No. | 0811320841-R010A4 |
| | | | SCALE N/A | REV. |
| DRAWN | LMK | July 06/10 | **FIGURE A-4** | |
| CHECK | | | | |

LDN_MTO_NEW GLDR_LDN.GDT



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|---|---|---|---|
| ● | 101 | 11 | 313.1 |
| ■ | 105 | 12 | 318.7 |

PROJECT
OTTAWA STREET SOUTH OVERPASS (Site 33-226)
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE

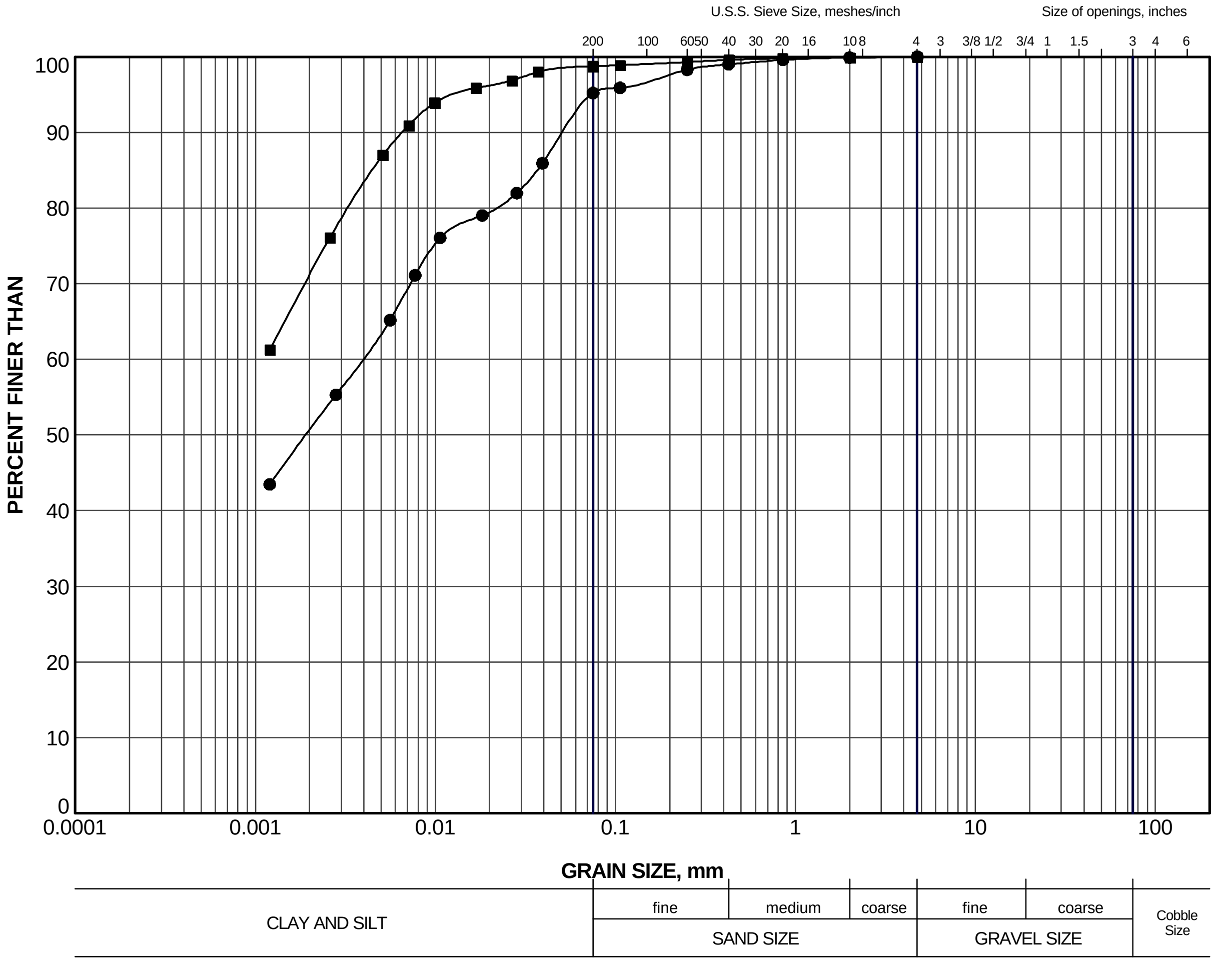
# GRAIN SIZE DISTRIBUTION

## SAND AND GRAVEL

Golder Associates
LONDON, ONTARIO

| | | | | |
|---|---|---|---|---|
| PROJECT No. | 08-1132-084-1 | | FILE No. | 0811320841-R010A5 |
| | | | SCALE N/A | REV. |
| DRAWN | DCH | July 06/10 | FIGURE A-5 | |
| CHECK | | | | |

LDN_MTO_NEW GLDR_LDN.GDT



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|---|---|---|---|
| ● | 105 | 15 | 314.2 |
| ■ | 105 | 19 | 308.1 |

PROJECT
OTTAWA STREET SOUTH OVERPASS (Site 33-226)
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE

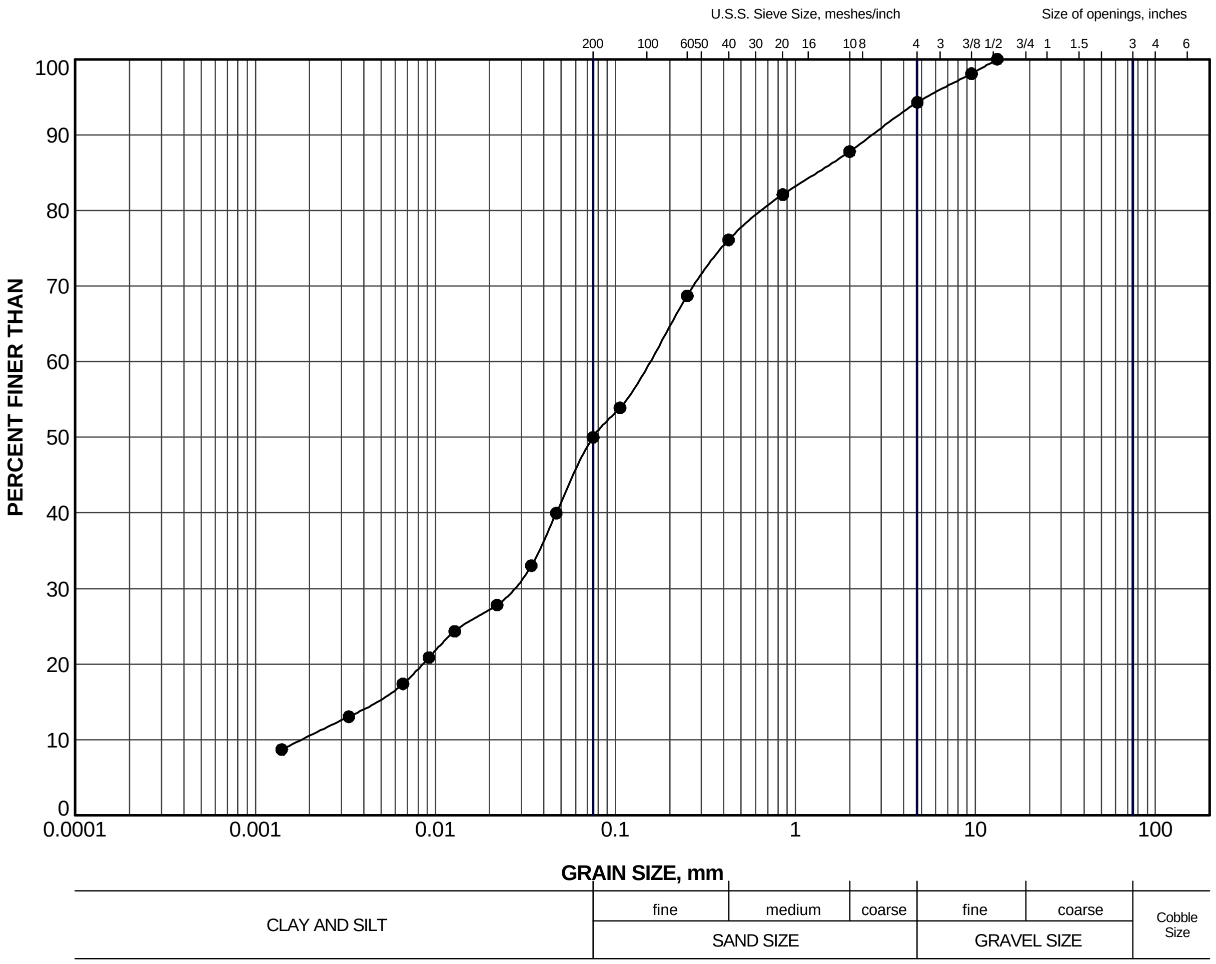
# GRAIN SIZE DISTRIBUTION

## SILTY CLAY TILL

Golder Associates
LONDON, ONTARIO

| | | | | |
|---|---|---|---|---|
| PROJECT No. | 08-1132-084-1 | | FILE No. | 0811320841-R010A6 |
| | | | SCALE N/A | REV. |
| DRAWN | DCH | July 06/10 | FIGURE A-6 | |
| CHECK | | | | |

LDN_MTO_NEW GLDR_LDN.GDT



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|---|---|---|---|
| ● | 105 | 22 | 303.6 |

PROJECT
OTTAWA STREET SOUTH OVERPASS (Site 33-226)
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE

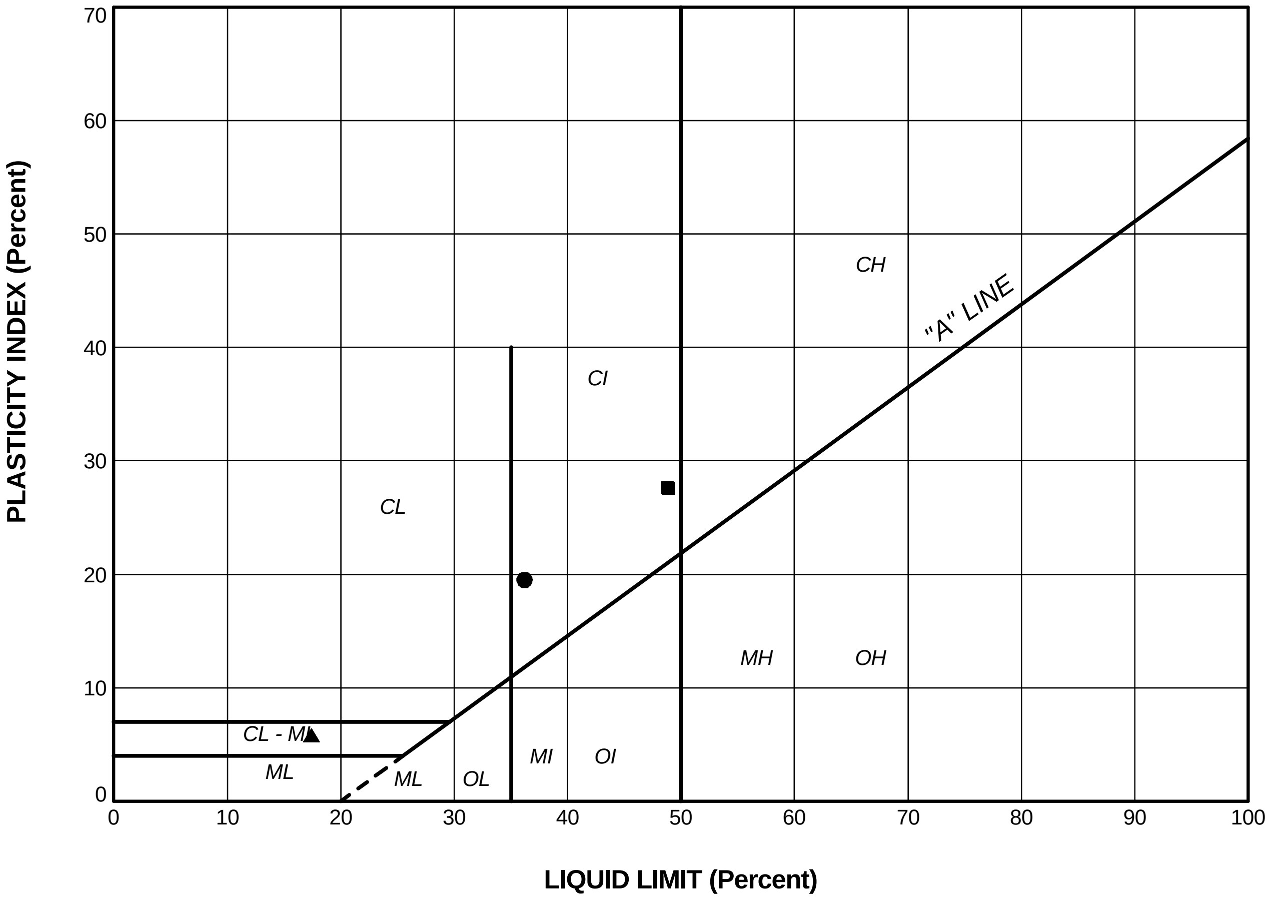
# GRAIN SIZE DISTRIBUTION

## SANDY SILT TILL

Golder Associates
LONDON, ONTARIO

| | | | | |
|---|---|---|---|---|
| PROJECT No. | 08-1132-084-1 | | FILE No. | 0811320841-R010A7 |
| | | | SCALE N/A | REV. |
| DRAWN | DCH | July 06/10 | **FIGURE A-7** | |
| CHECK | | | | |

LDN_MTO_NEW GLDR_LDN.GDT



**SOIL TYPE**
C = Clay
M = Silt
O = Organic

**PLASTICITY**
L = Low
I = Intermediate
H = High

## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | LL(%) | PL(%) | PI |
|---|---|---|---|---|---|
| SILTY CLAY TILL | | | | | |
| ● | 105 | 15 | 36.2 | 16.7 | 19.5 |
| ■ | 105 | 19 | 48.8 | 21.2 | 27.7 |
| SANDY SILT | | | | | |
| ▲ | 107 | 11 | 17.4 | 11.6 | 5.8 |

PROJECT
OTTAWA STREET SOUTH OVERPASS (Site 33-226)
WIDENING OF HIGHWAY 7/8
GWP 131-98-00

TITLE

# PLASTICITY CHART
## (SILTY CLAY TILL)

Golder Associates
LONDON, ONTARIO

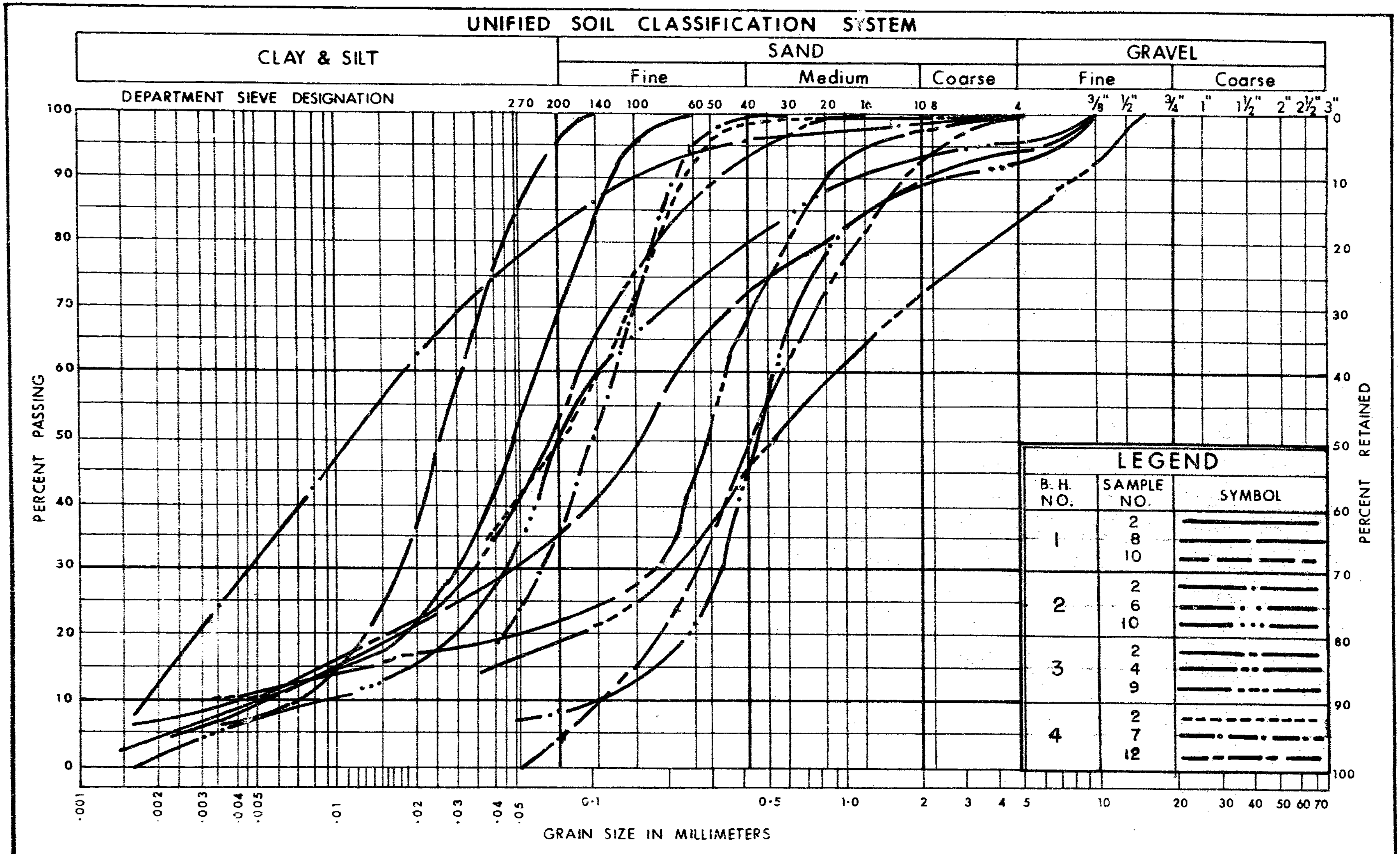
| | | | | | |
|---|---|---|---|---|---|
| PROJECT No. | | 08-1132-084-1 | FILE No. | 0811320841-R010A8 | |
| | | | SCALE | N/A | REV. |
| DRAWN | DCH | July 06/10 | FIGURE A-8 | | |
| CHECK | | | | | |

MTO_PI GLDR_LDN.GDT

# APPENDIX B

## Records of Previous Boreholes (Geocres Report No. 40P08-051)

FORM OB MT-17
REV. APR. 1965
65-414



**DEPARTMENT OF HIGHWAYS**
**MATERIALS and TESTING DIVISION**
ONTARIO

# GRAIN SIZE DISTRIBUTION

W.P. No. 626-64
JOB No. 66-F-69

FORM DB-MT-126 (FORMERLY DB-ML-126)
68-1831

OFFICE REPORT ON SOIL EXPLORATION

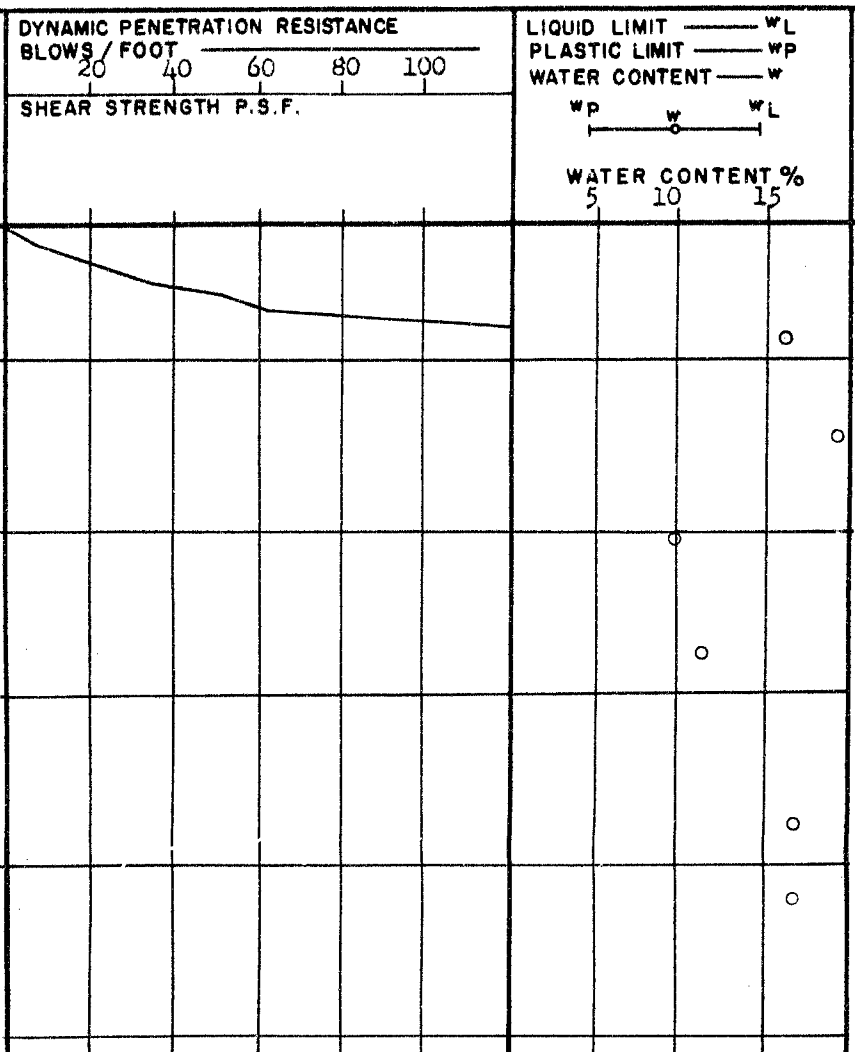
DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 66-F-69
W.P. 626-64
DATUM Geodetic
LOCATION Co-Ord's: N-187,867.5; E-208,135.4
BORING DATE July 27 - 29
BOREHOLE TYPE Washboring, NX & BX Casing
ORIGINATED BY A.M.S.
COMPILED BY A.M.S.
CHECKED BY K.G.S.

| SOIL PROFILE | | | SAMPLES | | | | | |
|---|---|---|---|---|---|---|---|---|
| ELEV. / DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | BLOWS / FOOT | ELEV. SCALE | BULK DENSITY γ P.C.F. | REMARKS |
| 1077.7 | Groundlevel | | | | | | | |
| 0 | | | | | | | | |
| | Fine Sandy Silt | | 1 | SS | 55 | | | |
| | To Silty Fine To | | 2 | SS | 74 | 1070 | | GrOSa31<br>Si64 Cl 5 |
| | Medium Sand With | | 3 | SS | 108 | | | |
| | Trace Of Gravel | | 4 | SS | 201/10" | | | |
| | | | 5 | SS | 142 | 1060 | | |
| | Very Dense | | 6 | SS | 130/5½" | | | |
| | | | 7 | SS | 100/6" | | | |
| | | | 8 | SS | 116/6" | 1050 | | Gr6Sa58<br>Si32C14 |
| 1047.7 | | | | | | | | W.L. ▼<br>1048.3 |
| 30.0 | Sandy Silt | | 9 | SS | 114/6" | | | |
| | To Silt With | | 10 | SS | 100/5" | 1040 | | GrOSa5<br>Si90C15 |
| | Trace Of Gravel | | | | | | | |
| 1036.2 | Very Dense | | 11 | SS | 100/4½" | | | |
| 41.5 | End Of Borehole | | | | | 1030 | | |



FORM 08-MT-126 (FORMERLY 08-ML-126)
65-1831

OFFICE REPORT ON SOIL EXPLORATION

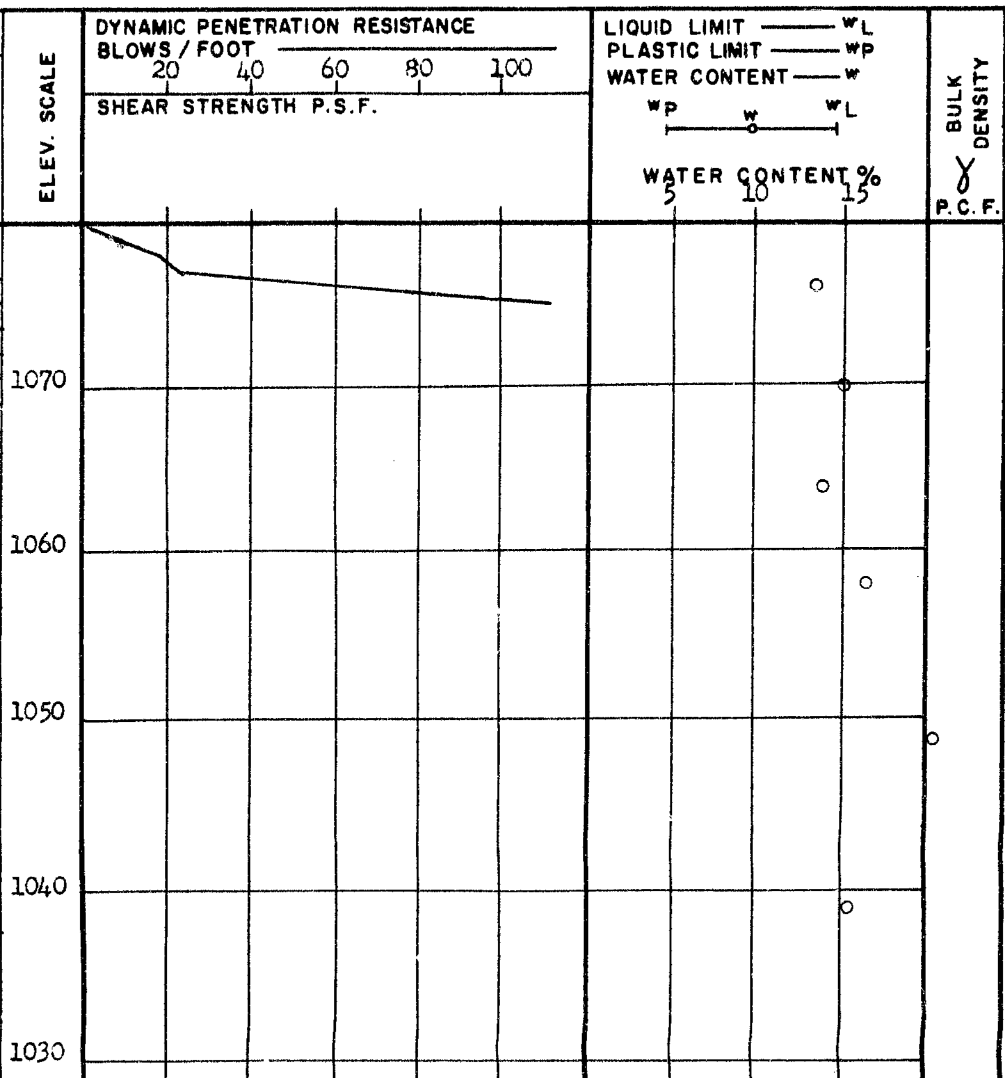
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 66-F-69
W.P. 626-64
DATUM Geodetic
LOCATION Co-Ord's: N-189,757.0; E-208,186.4
BORING DATE July 29 & August 2/66.
BOREHOLE TYPE Washboring & BX Casing
ORIGINATED BY AM.S.
COMPILED BY A.M.S.
CHECKED BY

| SOIL PROFILE | | | SAMPLES | | | | |
|---|---|---|---|---|---|---|---|
| ELEV. / DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | BLOWS / FOOT | BULK DENSITY γ P.C.F. | REMARKS |
| 1079.5 | Ground Level | | | | | | |
| 0 | Sandy Silt To | | | | | | |
| | Silty Sand With | | 1 | SS | 47 | | Gr0Sa17<br>Si72Cl 11 |
| | | | 2 | SS | 53 | | |
| | Traces of | | 3 | SS | 62 | | |
| | Gravel and | | 4 | SS | 58 | | |
| | Clayey Silt | | 5 | SS | 45 | | |
| | Compact To | | 6 | SS | 33 | | Gr7Sa85<br>SiC18 |
| | Very Dense | | 7 | SS | 25 | | |
| | | | 8 | SS | 24 | | |
| | | | 9 | SS | 24 | | W.L.<br>1049.4 |
| | | | 10 | SS | 44 | | Gr5Sa45<br>Si50C10 |
| 1038.0 | | | 11 | SS | 65 | | |
| 41.5 | End Of Borehole | | | | | | |



FORM DB-MT-126 (FORMERLY DB-ML-126)
68-1831

OFFICE REPORT ON SOIL EXPLORATION

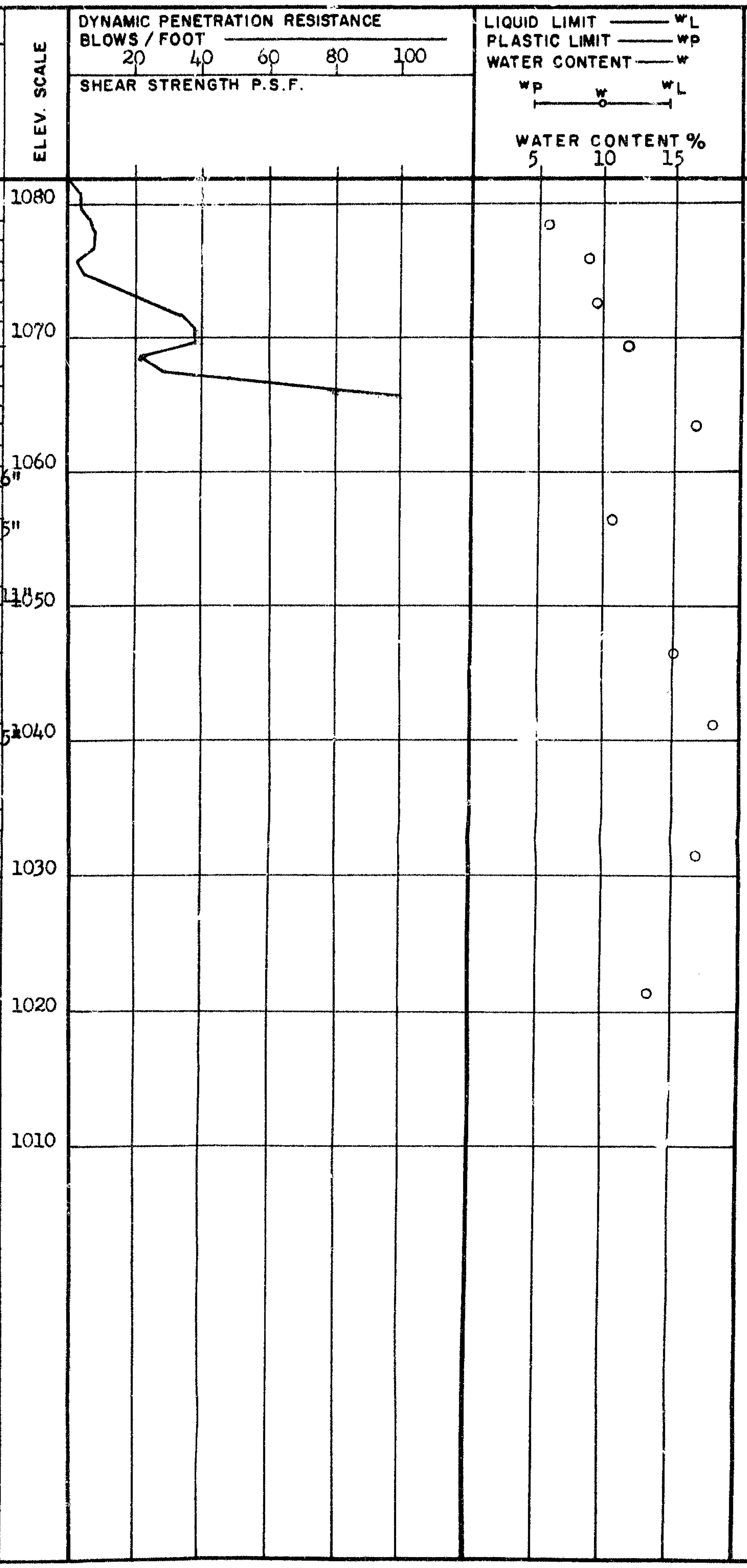
DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 66-F-69
W.P. 626-64
DATUM Geodetic
LOCATION Co-Ord's: N-189,894.5; E-208,261.8
BORING DATE August 3 & 4/66
BOREHOLE TYPE Washboring & BX Casing
ORIGINATED BY A.M.S.
COMPILED BY A.M.S.
CHECKED BY

| SOIL PROFILE | | | SAMPLES | | | BULK DENSITY γ P.C.F. | REMARKS |
|---|---|---|---|---|---|---|---|
| ELEV. / DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | BLOWS / FOOT | | |
| 1081.7 | Ground Level | | | | | | |
| 0 | Sandy Silt To Silty Sand With Trace Of Gravel Compact To Very Dense | | 1 | SS | 21 | | |
| | | | 2 | SS | 38 | | Gr0Sa46<br>Si47C17 |
| | | | 3 | SS | 101 | | |
| | | | 4 | SS | 56 | | Gr0Sa78<br>Si11Cl.11 |
| | | | 5 | SS | 166 | | |
| | | | 6 | SS | 145 | | |
| | | | 7 | SS | 100/6" | | |
| | | | 8 | SS | 106/5" | | |
| | | | 9 | SS | 172/11" | | Gr16Sa65<br>SiCl.19 |
| | | | 10 | SS | 102 | | W.L.<br>1047.8 |
| | | | 11 | SS | 100/5" | | |
| | | | 12 | SS | 120 | | |
| 1031.7<br>50.0 | Silty Clay To Clayey Silt With Occasional Seams and Pockets Of Silt, Trace of Sand and Gravel Hard | | 13 | SS | 98 | | |
| | | | 14 | SS | 186 | | |
| 1020.2<br>61.5 | End Of Borehole | | | | | | |



FORM DB-MT-126 (FORMERLY DB-ML-126)
68-1831

OFFICE REPORT ON SOIL EXPLORATION

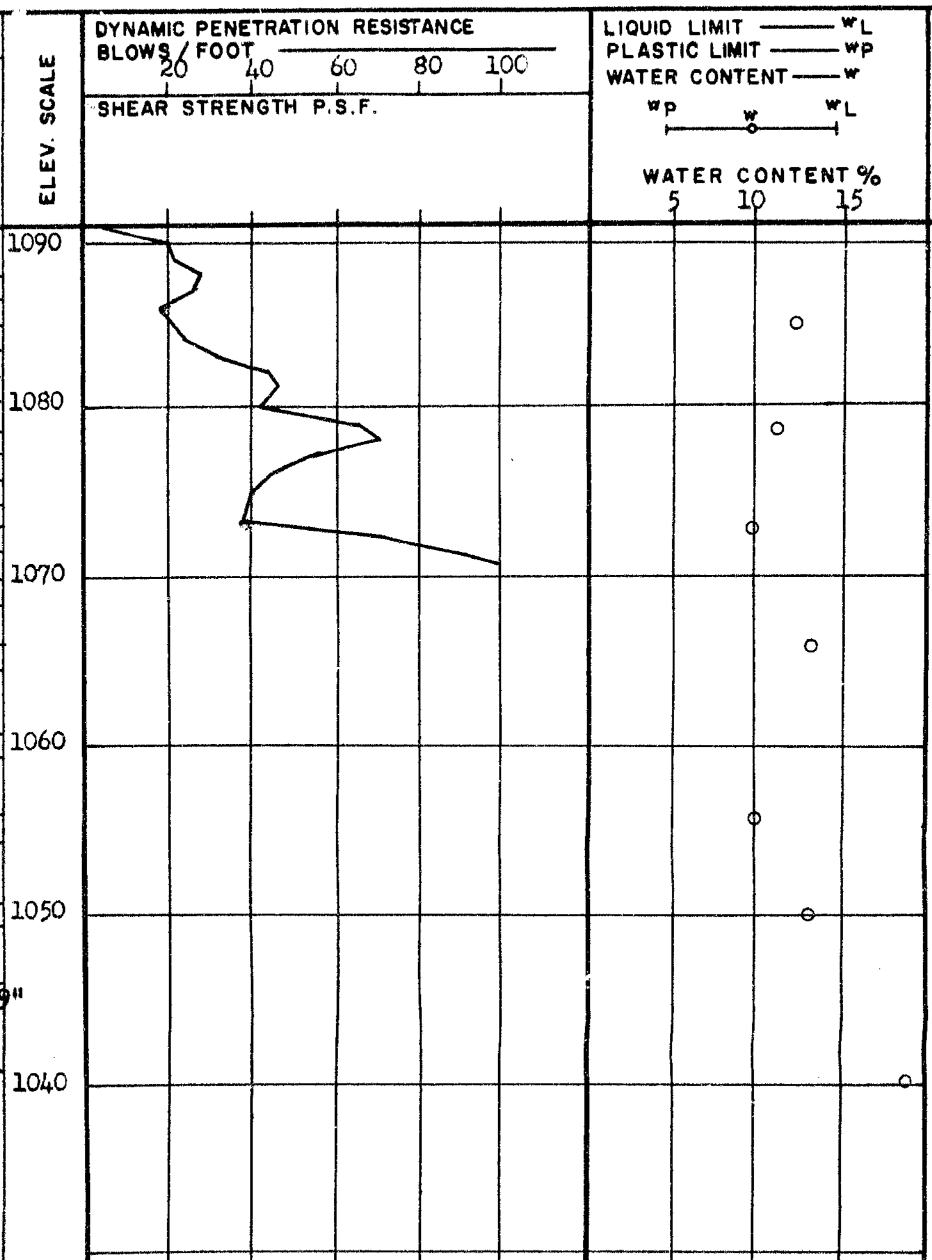
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 66-F-69
W.P. 626-64
DATUM Geodetic
LOCATION Co-Ord's: N-189,784.0; E-208,303.4
BORING DATE August 4 & 5/66
BOREHOLE TYPE Washboring & BX Casing
ORIGINATED BY A.M.S.
COMPILED BY A.M.S.
CHECKED BY

| ELEV. / DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | BLOWS / FOOT | BULK DENSITY γ P.C.F. | REMARKS |
|---|---|---|---|---|---|---|---|
| 1091.1 | Ground Level | | | | | | |
| | Sandy Silt To Silty Sand With Traces Of Gravel And Clayey Silt Compact To Very Dense | | 1 | SS | 26 | | |
| | | | 2 | SS | 36 | | Gr0Sa50<br>SiCl50 |
| | | | 3 | SS | 56 | | |
| | | | 4 | SS | 38 | | |
| | | | 5 | SS | 31 | | |
| | | | 6 | SS | 114 | | |
| | | | 7 | SS | 106 | | Gr0Sa64<br>SiCl36 |
| | | | 8 | SS | 80 | | |
| | | | 9 | SS | 136 | | |
| | | | 10 | SS | 102 | | |
| | | | 11 | SS | 139 | | W.L.<br>1053.0 |
| 1046.1<br>45.0 | Fine To Medium Sand With Traces of Silt Very Dense | | 12 | SS | 182/9" | | Gr0Sa95<br>SiCl5 |
| 1041.1<br>50.0 | End Of Borehole | | | | | | |



# APPENDIX C

## Site Photographs

Photo 1: Ottawa Street South overpass, looking south from Imperial Drive.

Photo 2: Ottawa Street South overpass, looking north from S-E ramp.

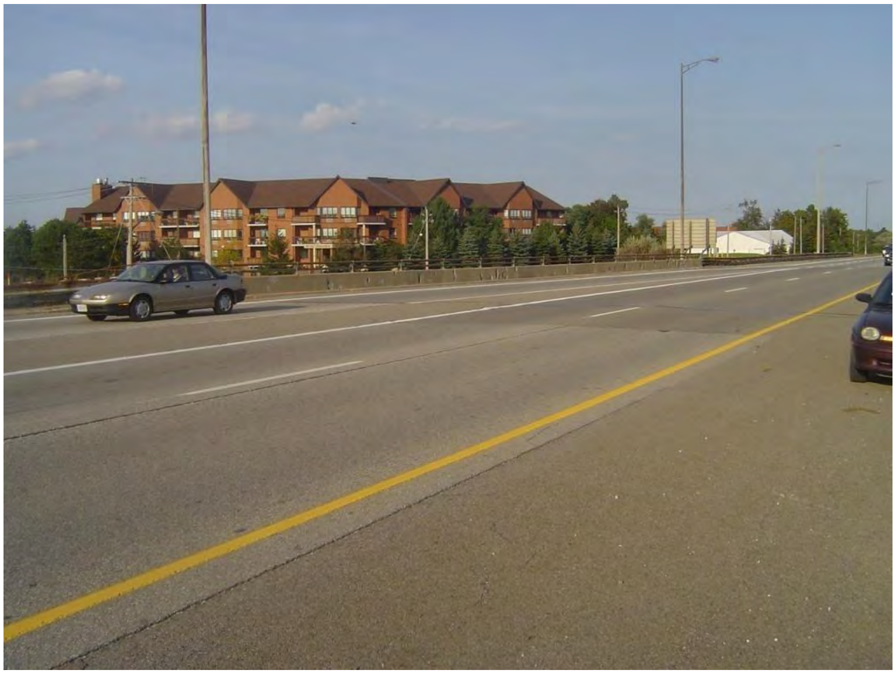


Photo 3: Westbound lanes of Ottawa Street South overpass.

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

| | |
|---|---|
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| Asia | + 852 2562 3658 |
| Australasia | + 61 3 8862 3500 |
| Europe | + 356 21 42 30 20 |
| North America | + 1 800 275 3281 |
| South America | + 55 21 3095 9500 |

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**T: +1 (519) 652 0099**