



August 10, 2017

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

**DUNLOP STREET UNDERPASS, SITE NO. 30-175
HIGHWAY 400 WIDENING
FROM 1 KM SOUTH OF HIGHWAY 89 TO JUNCTION OF HIGHWAY 11
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 06-20016**

Submitted to:

AECOM
30 Leek Crescent, 4th Floor
Richmond Hill, Ontario
L4B 4N4



REPORT

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HIGHWAY 400 DUNLOP STREET UNDERPASS**

PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
DUNLOP STREET UNDERPASS – SITE NO. 30-175
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (now AECOM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the replacement of the Dunlop Street Underpass (Site No. 30-175) in the City of Barrie. The proposed work is part of the preliminary and design-build ready design associated with the Highway 400 widening from 1 km south of Highway 89 to the junction of Highway 11 in Simcoe County, Ontario.

This report addresses the proposed replacement of the Dunlop Street Underpass (MTO Structure Site No. 30-175) and the associated approach embankments only.

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated July 2013. Golder's scope of work for foundation engineering services associated with the Dunlop Street Underpass replacement is contained in Section 5.8 of AECOM's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated January 20, 2014.

2.0 SITE DESCRIPTION

The Dunlop Street Underpass, which is part of the Highway 400-Dunlop Street (Simcoe Road 90, formerly Highway 90) Interchange, is located approximately 2.3 km north of Essa Road and south of Bayfield Street, in Barrie, Ontario, as shown in the Key Plan on Drawing 1. Dunlop Street cross Highway 400 on an approximately 38 degree skew oriented southwesterly-northeasterly relative to Highway 400. The overall surface topography in the vicinity of the site is relatively flat and land use consists of commercial businesses to the east and west of Highway 400.

At Dunlop Street, the Highway 400 grade is near original ground surface at about Elevation 231 m and the Dunlop Street approach embankments are about 6.5 m high with the roadway surface at about Elevation 237.5 m. The existing bridge is a two-span structure supported on spread footings. The existing bridge deck is approximately 79.5 m long, with span lengths of about 27 m and a deck width of about 14 m, as measured parallel to Highway 400.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Borehole Investigation

One borehole was advanced at this site as part of a previous investigation carried out by Golder Associates in 2001 (MTO, 2002) for the replacement of the existing Dunlop Street Underpass structure, associated with the widening of Highway 400. Borehole B11-1 was advanced on the west side of Highway 400, north of Dunlop Street from the Highway 400 grade, to a depth of 39.8 m below ground surface. The borehole location is shown on Drawing 1 and the Record of Borehole sheet and associated laboratory testing results are provided in Appendix A.

The borehole was advanced using NW and BW size casing. Samples of the overburden were obtained at intervals of depth of about 0.75 m and 1.5 m using a 50 mm outer diameter split-spoon sampler driven by a manual hammer in accordance with the Standard Penetration Test (SPT) procedure.



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The water level in the open borehole was observed during and following the drilling operations and a piezometer was installed in the borehole to allow monitoring of the groundwater level at the site.

The borehole location in MTM NAD83 (Zone 10) northing and easting coordinates, ground surface elevations reference to Geodetic datum and drilled depth are summarized below.

Borehole Number	Location (MTM NAD83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
BH11-1	4,915,592.6	287,961.2	231.2	39.8

3.2 Current Borehole Investigation

The field work at the site of the Dunlop Street Underpass was carried out between November 27 and 30, 2016 during which time one borehole (designated Borehole DL1-1) was advanced to supplement the existing subsurface information from the 2001 investigation. In addition, a Dynamic Cone Penetration Test (DCPT) was advanced from the bottom of Borehole DL1-1. The Record of Borehole DL1-1 is presented in Appendix B. The location of this borehole is shown in plan on Drawing 1 and in profile and cross-section on Drawings 2 and 3.

The borehole investigation was carried out using a Diedrich D-90 truck-mounted drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The borehole was advanced through the overburden using 108 mm inside diameter hollow stem augers and NW size casing with wash boring techniques. Soil samples were generally obtained at intervals of depth about 0.75 m and 1.5 m, using a 50 mm outside diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586). The groundwater conditions and water level in the open boreholes were observed during and immediately following the completion of drilling operations and a piezometer was installed in a borehole adjacent to Borehole DL1-1 to allow monitoring of the groundwater level at the site. During the extraction of the NQ casing (71 mm O.D.) in Borehole DL1-1 a casing connection sheared resulting in about 24.4 m of NQ casing remaining in the borehole, between depths of about 21.8 m and 46.2 m below ground surface, corresponding to between Elevations 209.8 m and 185.4 m. Attempts to retrieve the casing were unsuccessful and as a result Borehole DL1-1 and the 24.4 m length of NQ casing were backfilled with bentonite pellets.

The field work was observed by a member of Golder's engineering staff who located the borehole, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the borehole and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, grain size distribution and Atterberg limits) was carried out on selected soil samples. The results of the laboratory testing are presented on the Record of Borehole sheet and are shown on the laboratory test sheets included in Appendix B.

The as-drilled borehole location was measured relative to the existing on-site features shown on the Digital Terrain Model (DTM) for the site, and the ground surface elevation was interpolated from the topographic data provided by AECOM. The borehole location provided on the Record of Borehole and presented on Drawings 1 to 3 are given using MTM NAD83 (Zone 10) northing and easting coordinates, and the ground surface elevation is referenced to Geodetic datum. The borehole location, ground surface elevation and drilled depth are summarized below.



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Borehole Number	Location (MTM NAD83)		Ground Surface Elevation (m)	Borehole / DCPT Depth (m)
	Northing (m) / Latitude (°)	Easting (m) / Longitude (°)		
DL1-1	4,915,644.7 / 44.380615	288,037.8 / -79.710359	231.6	46.2 / 47.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*¹, the section of Highway 400 extending from 6 km south of Highway 89 to the junction of Highway 11 traverses, generally in a south–north direction, the following physiographic regions: the Peterborough Drumlin Field; the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Peterborough Drumlin Field is present from the southern limit of the project site to south of Line 13 of the Township of Bradford West Gwillimbury, as well as between about 1 km north of Highway 89 to about Essa Road. The Simcoe Lowlands covers the area from south of Line 13 to approximately 1 km north of Highway 89 and from about Essa Road to just north of Anne Street. The Simcoe Uplands extends from just north of Anne Street to beyond the northern limit of the project site.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Deposits of silt, clay or peat may also be found in the low-lying areas between drumlins and eskers.

Along Highway 400, the Simcoe Lowlands include: the Holland River valley; the lowlands of the Lake Simcoe basin to the east; and the lowlands of the Nottawasaga basin to the west, which includes Innisfil Creek and the Nottawasaga River to the south and west of the project limits, respectively. The Lake Simcoe and Nottawasaga basins are connected by a flat floored valley through Barrie which extends from the shores of Kempenfelt Bay west generally along Highway 90. The Simcoe Lowlands are generally characterized by deep deposits of deltaic or lacustrine silts, sands and clays associated with glacial Lake Algonquin.

The Simcoe Uplands consist of till plains and ancient shorelines. The till deposits range from clayey to silty and generally become sandier and containing more boulders in the north section. The low-lying areas of this region may also contain shallow deposits of sand and gravel associated with former glacial lake shorelines.

4.2 Subsurface Conditions

The Record of Borehole sheets and laboratory testing results from the 2001 and 2016 investigations are presented in Appendices A and B, respectively. The interpreted stratigraphic profile and cross-sections are shown on Drawings 2 and 3.

The results of the in situ field tests (i.e. SPT 'N'-values) carried out during the 2001 and 2016 investigations as presented on the Record of Borehole sheets and in Section 4.2 are uncorrected. According to the Canadian Foundation Engineering Manual (*CFEM*, 2006), the energy delivered to the drill rod varies with the hammer release system, hammer type, anvil and operator characteristics, and as different hammer release systems were

¹ Chapman, L. J. and Putnam, D. F., 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey. Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000. Ontario Ministry of Natural Resources.



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used during the 2001 and 2016 investigations (i.e. manual vs automatic, respectively) the SPT 'N'-values measured during the 2001 investigation may be higher than the 'N'-values measured during the 2016 investigation within the same deposit. In addition, some soil descriptions from the previous investigation differ slightly from the descriptions associated with the laboratory testing results. For the purposes of this report the soils are described consistent with the test results (i.e. for Atterberg limits).

The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profile and cross-sections are inferred from observations of drilling progress and non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

4.2.1 West Abutment, Pier 1 and Pier 2

Borehole B11-1 was advanced between the proposed south pier and central pier (Pier 1 and Pier 2). In general, the subsurface conditions at the western section of the structure consist of a layer of topsoil and silty clay fill underlain by alternating deposit of clayey silt, sandy silt and silty sand. A detailed description of the subsurface conditions encountered at this area of the site are provided in the following sections.

4.2.1.1 Topsoil

A 200 mm thick layer of topsoil was encountered at surface in Borehole B11-1.

4.2.1.2 Silty Clay Fill

A 0.6 m thick deposit of silty clay fill was encountered below the topsoil in Borehole B11-1 at Elevation 231.0 m.

4.2.1.3 Clayey Silt

A 9.9 m thick deposit of clayey silt was encountered below the fill in Borehole B11-1 at Elevation 230.4 m and pockets of silty sand up to 0.4 m thick. An interlayer of silty sand about 1.7 m thick were encountered within the clayey silt deposit at depths of 4.1 m and 6.6 m, corresponding to Elevations 227.1 m and 224.6 m, respectively.

The SPT 'N'-values measured within the clayey silt deposit range from 4 blows to 19 blows per 0.3 m of penetration, suggesting a firm to very stiff consistency. The SPT 'N'-value measured at the boundary of the clayey silt deposit and the underlying silty sand interlayer is 26 blows per 0.3 m of penetration, suggesting a very stiff consistency transitioning to a compact relative density. An SPT 'N'-value measured within the silty sand interlayer is 37 blows per 0.3 m of penetration, indicating a dense relative density.

The natural water content measured on four samples of the clayey silt deposit are between about 15 per cent and 20 per cent. The natural water content measured on a sample of the silty sand interlayer is about 16 per cent.

The result of a grain size distribution test completed on one sample of the clayey silt deposit is provided on Figure 1 in Appendix A.



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Atterberg limits test carried out on three samples of the cohesive deposit measured liquid limits between about 22 per cent and 27 per cent, plastic limits between about 13 per cent and 16 per cent and plasticity indices between about 7 per cent and 12 per cent. The results of the Atterberg limits tests indicate that the material is classified as a clayey silt of low plasticity.

4.2.1.4 *Sandy Silt*

A 7.3 m thick deposit of sandy silt with silty clay interlayers was encountered below the clayey silt deposit in Borehole B11-1 at Elevation 220.5 m.

The SPT 'N'-values measured within the sandy silt deposit range from 31 blows to 48 blows per 0.3 m of penetration, indicating a dense relative density.

The natural water content measured on three samples of the sandy silt deposit range between about 14 per cent and 17 per cent.

The result of the grain size distribution test completed on a sample of the sandy silt deposit is provided on Figure 2 in Appendix A.

4.2.1.5 *Silty Clay*

A 20.7 m thick deposit of silty clay was encountered below the sandy silt deposit in Borehole B11-1 at Elevation 213.2 m. A 0.3 m thick silty sand and a sandy silt pocket was encountered within the silty clay deposit at depths of 19.8 m and 29.3 m, corresponding to Elevations 211.4 m and 201.9 m, respectively.

The SPT 'N'-values measured within the silty clay deposit range from 15 blows to 54 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

The natural water content measured on three samples of the silty clay deposit are between about 16 per cent and 24 per cent.

4.2.1.6 *Silty Sand*

A silty sand deposit was encountered below the silty clay deposit in Borehole 11-1 at Elevation 192.5 m and penetrated into for 1.1 m before borehole termination.

The SPT 'N'-value measured within silty sand deposit is 175 blows per 0.15 m of penetration, indicating a very dense relative density.

The natural water content measured on a sample of the silty sand deposit is about 22 per cent.

4.2.2 *Pier #3 and East Abutment*

Borehole DL1-1 was advanced near the proposed north pier (Pier 3) and East Abutment. In general, the subsurface conditions in this area of the site consists of a layer of topsoil and sand fill underlain by alternating



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deposit of clayey silt, silt, silty sand, silt and sand to sand. A detailed description of the subsurface conditions encountered at this area of the site are provided in the following sections.

4.2.2.1 Topsoil

A 200 mm thick layer of topsoil was encountered at ground surface in Borehole DL1-1

4.2.2.2 Sand Fill

A 2.0 m thick deposit of sand fill was encountered below the topsoil in Borehole DL1-1 at Elevation 231.4 m.

The SPT 'N'-values measured within the sand fill range from 13 blows to 48 blows per 0.3 m of penetration, indicating a compact to dense relative density.

The natural water content measured on a sample of the sand fill is about 12 per cent.

4.2.2.3 Sand

A 6.5 m thick deposit of sand was encountered below the fill in Borehole DL1-1 at Elevation 229.4 m.

The SPT 'N'-values measured within the sand deposit range from 5 blows to 46 blows per 0.3 m of penetration, indicating a loose to dense relative density.

The natural water content measured on two samples of the sand deposit are about 22 per cent and 25 per cent.

The result of a grain size distribution test completed on one sample of the sand deposit is provided on Figure B1 in Appendix B.

4.2.2.4 Silt (Upper)

An upper silt deposit 18.9 m thick was encountered below the sand deposit in Borehole DL1-1 at Elevation 222.9 m. Sand pockets and interlayers about 0.6 m and 1.4 m thick were encountered within the upper silt deposit at Elevations 218.9 m and 205.4 m, respectively.

The SPT 'N'-values measured within the upper silt deposit range from 16 blows to 37 blows per 0.3 m of penetration, indicating a compact to dense relative density.

The natural water content measured on six samples of the upper silt deposit range between about 15 per cent and 21 per cent.

The results of the grain size distribution tests completed on three sample of the upper silt deposit from are provided on Figure B2 in Appendix B.

Atterberg limits test carried out on two samples of the upper silt deposit measured liquid limits of about 18 per cent, plastic limits of about 15 per cent and corresponding plasticity indices of about 3 per cent. The results of the Atterberg limits tests, which are shown on Figure B3 in Appendix B, indicate that the material is



classified as a silt of slight plasticity. The result of one Atterberg limits test carried out on a sample of the upper silt deposit indicate that the material is non-plastic.

4.2.2.5 *Clayey Silt*

A 3.2 m thick deposit of clayey silt was encountered below the sand interlayer within the silt deposit in Borehole DL1-1 at Elevation 204.0 m.

The SPT 'N'-values measured within the clayey silt deposit are 31 blows per 0.3 m of penetration, indicating a hard consistency.

The natural water content measured on a sample of the clayey silt deposit is about 18 per cent.

An Atterberg limits test carried out on a sample of the clayey silt deposit measured a liquid limit of about 20 per cent, a plastic limit of about 12 per cent and a corresponding plasticity index of about 8 per cent. The result of the Atterberg limits test, which is shown on Figure B4 in Appendix B, indicates that the material is classified as a clayey silt of low plasticity.

4.2.2.6 *Silt (Lower)*

A lower silt deposit 10.7 m thick deposit containing silty clay lenses was encountered below the clayey silt in Borehole DL1-1 at Elevation 200.8 m.

The SPT 'N'-values measured within the lower silt deposit range from 22 blows to 64 blows per 0.3 m of penetration, indicating a compact to very dense relative density.

The natural water content measured on three samples of the silt deposit are between about 19 per cent and 23 per cent.

The result of the grain size distribution test completed on one sample of the lower silt deposit from Borehole DL1-1 is shown on Figure B5 in Appendix B.

An Atterberg limits test carried out on a sample of the silt deposit measured a liquid limit of about 19 per cent, a plastic limit of about 16 per cent and corresponding plastic index of about 3 per cent. The result of the Atterberg limits test, which is shown on Figure B6 in Appendix B, indicates that the material is classified as a silt of slight plasticity, and an Atterberg limits test carried out on another sample of the lower silt deposit indicates that the material is non-plastic.

4.2.2.7 *Silt and Sand to Silty Sand*

A silt and sand to silty sand deposit, comprised of an upper layer of silty sand and an underlying layer of silt and sand, was encountered below the lower silt deposit in Borehole DL1-1 at Elevation 190.1 m and was penetrated into for a depth of 4.7 m before borehole termination. The deposit thickness may potentially be greater than 5.9 m as inferred from a DCPT advanced from the bottom of Borehole DL1-1.

The SPT 'N'-values measured within the silt and sand to silty sand deposit are 95 blows and 157 blows per 0.3 m of penetration, indicating a very dense relative density.



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The natural water content measured on a sample of the silt and sand portion of the deposit is about 18 per cent.

The result of the grain size distribution test completed on one sample of the silt and sand portion of the deposit is shown on Figure B7 in Appendix B.

4.3 Groundwater Conditions

The water level encountered during drilling and observed in Borehole B11-1 upon completion of drilling in 2001 was between approximately Elevation 225.0 m and 222.4 m.

Standpipe piezometers were installed in Borehole B11-1 and DL1-1 as part of the 2001 and 2016 investigations, respectively. The observed groundwater level in the standpipe piezometers is shown on the Record of Borehole sheets and summarized below:

Borehole	Depth to Water Level (m)	Groundwater Elevation (m)	Date of Measurement
B11-1	1.3	229.9	March 15, 2001
DL1-1	1.9	229.7	March 15, 2017

The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and during periods of precipitation.

5.0 CLOSURE

This report was prepared by Ms. Amelia Jewison, B.A.Sc., EIT, a member of the geotechnical engineering group, and was reviewed by Mr. Christopher Ng, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., a Senior Consultant with Golder and Designated MTO Foundations Contact, conducted an independent quality control review of this report.

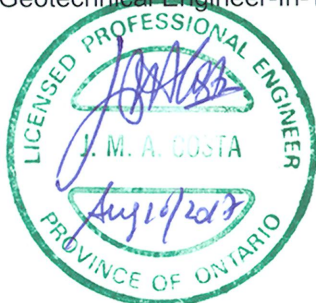


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Report Signature Page

GOLDER ASSOCIATES LTD.

Amelia Jewison, B.A.Sc., EIT,
Geotechnical Engineer-In-Training



Jorge M. A. Costa, P.Eng.
Designated MTO Foundations Contact, Senior Consultant



Christopher Ng, P.Eng.
Senior Geotechnical Engineer, Associate

AJ/MCK/CN/JMAC/mck

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PART B

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6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the Highway 400-Dunlop Street Underpass (MTO Structure Site No. 30-175). These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current and a previous subsurface investigation. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the feasible foundation alternatives and carry out the design of the structure foundations, as may be required. The Foundation Investigation Report, discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part A of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the MTO to provide recommendations on foundation aspect for the preliminary design of the Highway 400-Dunlop Street Underpass in the City of Barrie.

Based on the General Arrangement (GA) Drawing provided by AECOM on June 23, 2016, It is understood that the Dunlop Street Underpass will consist of an approximately 37 m wide, 154.5 m long, four-span bridge with approximate span lengths between about 29.5 m and 47.5 m. The existing approach embankment will be widened by about 11.5 m and 13.5 m to the north and south to accommodate the new under structure. In addition, a grade raise of about 2.3 m and 3.3 m will be required at the west and east abutments, respectively. The grade of the proposed Underpass varies between about Elevations 238.4 m and 240.1 m at the west and east abutments, respectively. In comparison, the proposed grade for Highway 400 below the proposed Underpass is at about Elevation 231.5 m.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 Canadian Highway Bridge Design Code (2014 CHBDC) and its Commentary, the proposed underpass structure and foundation system may be classified as having large traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a “typical consequence level” associated with exceeding limits states design. In addition, given the limited level of foundation investigation completed to date as presented in Sections 3.0 and 4.0, in comparison to the degree of site understanding in Section 6.5 of *CHBDC (2014)*, the level of confidence for design is considered to be a “low degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ , and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Tables 6.1 and 6.2 of the CHBDC have been used for design, as indicated in Sections 6.4 to 6.8.



6.3 Foundation Options

As part of the future widening of Highway 400 in Simcoe County, the existing Dunlop Street Underpass will require replacement. According to the available information, the existing two-span structure is supported on spread footings that are founded at approximately Elevation 229.0 m. Highway 400 is proposed to be widened by approximately 45 m to the west and 30 m to the east from the edge of the existing highway. Based on the proposed Underpass geometry and the subsurface conditions at this site, both shallow foundation and deep foundation options have been considered for support of the abutments and piers for the proposed structure. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the foundation alternative for a new (replacement) structure is presented in Table 1.

- **Shallow foundations – spread/strip footings:** Shallow foundations comprised of spread or strip footings, founded on either firm to very stiff clayey silt deposit, compact sand deposits or on a compact granular pad, are feasible for support of the new abutments and piers, although low factored geotechnical resistances are available; and this foundation type will preclude the use of integral abutments.
- **Deep foundations – driven steel H-piles or pipe (tube) piles:** Driven steel H-piles or steel pipe (tube) piles are feasible for support of the abutments and piers, and would permit design of conventional abutments, semi-integral abutments (for H-piles and pipe piles) or integral abutments (for H-piles only).
- **Deep foundations – drilled shaft (caissons):** Drilled shafts (caissons) are considered feasible for the support of the abutments and piers; however this option would preclude integral abutment design. This option would be more expensive than either shallow foundations or driven pile foundations, although fewer caisson elements would be required in comparison to the number of driven steel piles that would be required. If caissons are adopted for support of the abutments and piers temporary liners may be required during construction to control potential ground losses and/or disturbance of the caisson base.

Based on the above considerations and as detailed in Table 1, both shallow and deep foundation options are considered feasible for the support of the new abutments and piers.

6.4 Shallow Foundations

6.4.1 Founding Elevation

Founding Elevation for the support of the new abutments and piers spread/strip footings should be founded on firm to very stiff clayey silt deposit or, compact sand deposits, or on compacted granular pads. The highest founding elevations recommended for preliminary design of footings are:



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Foundation Element	Highest Founding Elevation (m)	Founding Soil
West Abutment	228.5	Firm to Very Stiff Clayey Silt
	227.5 ¹	Compacted Granular 'A'
Pier No. 1	228.5	Firm to Very Stiff Clayey Silt
	227.5 ¹	Compacted Granular 'A'
Pier No. 2	228.5	Firm to Very Stiff Clayey Silt
	227.5 ¹	Compacted Granular 'A'
Pier No. 3	229.0	Compact Sand
	228.0 ¹	Compacted Granular 'A'
East Abutment	229.0	Compact Sand
	228.0 ¹	Compacted Granular 'A'

Note: 1. Highest founding elevation of the Compacted Granular 'A' pad.

6.4.2 Factored Geotechnical Resistances

The following factored ultimate and serviceability geotechnical resistances (at ULS and SLS for 25 mm of settlement, respectively) may be used for preliminary design of spread/strip footings founded on the properly prepared soil, or on a compacted Granular 'A' pad having a minimum thickness of 1 m:

Foundation Elements	Foundation Alternative	Factored Ultimate Geotechnical Resistance ¹ (at ULS)	Factored Serviceability Geotechnical Resistance ¹ (at SLS) for 25 mm of Settlement
West Abutment, Pier 1 and Pier 2	Footing on properly prepared firm to very stiff clayey silt	275 kPa	150 kPa
	Footing on minimum 1 m thick compacted Granular 'A' pad	450 kPa	200 kPa
East Abutment and Pier 3	Footing on properly prepared loose sand	350 kPa	100 kPa
	Footing on minimum 1 m thick compacted Granular 'A' pad	400 kPa	100 kPa

Note: 1. The factored geotechnical resistances given above are estimated for a 3 m wide spread/strip footing.

The preliminary factored geotechnical resistances provided above are given for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Section 6.10.4 of the *CHBDC (2014)*.

6.4.3 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between cast-in-place concrete footings and the founding soils should be calculated in accordance with Section 6.10.5 of the *CHBDC (2014)*. The following presents the



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coefficient of friction, $\tan \delta$, for the interface between the concrete footing and native soil or fill deposit or Granular 'A' pad as interpreted from NAVFAC (1984):

Founding Material	Coefficient of Friction ($\tan \delta$)
Cast-in-place concrete footing on firm to very stiff clayey silt	0.35
Cast-in-place concrete footing on loose sand	0.45
Cast-in-place concrete footing on compacted Granular 'A' pad	0.60

6.4.4 Frost Protection

All footings should be provided with a minimum 1.5 m thick layer of soil cover for frost protection as per OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*), as measured vertically from ground surface and perpendicular to the face of the abutment slope to the edge of the underside of the footing.

If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.5 Driven Steel H-Pile or Steel Pipe (Tube) Pile Foundations

6.5.1 Founding Elevation

The abutments and piers for the replacement structure may be supported on steel H-piles or pipe piles driven to found within the very dense silty sand or silt and sand deposits.

Based on the GA Drawing, semi-integral abutments are proposed to be adopted for the design of the replacement structure with the abutments "perched" within the Dunlop Street embankments. Pile cap elevations equal to or higher than those provided below are acceptable for design from a foundations perspective as long as adequate soil cover is provided to the pile caps for frost protection. The following pile tip elevations for the abutments and piers could be considered for preliminary design purposes, using 1.5 m to 2 m penetration into the deposit exhibiting greater than 100 blows per 0.3 m penetration:

Foundation Element	Proposed Underside of Pile Cap	Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation
West Abutment	233 m	200 m	Very Stiff to Hard Silty Clay
		190 m	Very Dense Silty Sand
Pier 1	227.5 m	200 m	Very Stiff to Hard Silty Clay
		190 m	Very Dense Silty Sand
Pier 2	228.5 m	200 m	Very Stiff to Hard Silty Clay
		190 m	Very Dense Silty Sand



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Foundation Element	Proposed Underside of Pile Cap	Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation
Pier 3	229.5 m	200 m	Dense Silt
		185 m	Very Dense Silty Sand / Silt and Sand
East Abutment	234 m	200 m	Dense Silt
		185 m	Very Dense Silty Sand / Silt and Sand

6.5.2 Factored Geotechnical Axial Resistances

The factored ultimate and serviceability geotechnical axial resistances (at ULS and SLS for 25 mm of settlement, respectively) for driven steel H-piles and closed-end, concrete-filled 324 mm (12¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.) are presented below.

Foundation Element	Pile Type	Estimated Design Tip Elevation	Factored Ultimate Geotechnical Axial Resistance (at ULS)	Factored Serviceability Geotechnical Resistance (at SLS) for 25 mm of Settlement ¹
West Abutment, Pier 1 and Pier 2	HP 310x110	200 m	1,250 kN	N/A
		190 m	1,550 kN	N/A
	324 mm OD Pipe Pile	200 m	1,000 kN	N/A
		190 m	1,150 kN	N/A
East Abutment and Pier 3	HP 310x110	200 m	1,200 kN	N/A
		185 m	1,700 kN	N/A
	324 mm OD Pipe Pile	200 m	1,100 kN	N/A
		185 m	1,350 kN	N/A

Note: 1. The factored serviceability geotechnical resistance (at SLS) for 25 mm of settlement will be greater than the factored ultimate geotechnical axial resistance (at ULS) and as such, the SLS condition does not apply.

Pile installation should be in accordance with OPSS 903 (Deep Foundations). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO's Standard Drawing SS103-11, *Pile Driving Control*) and/or Pile Dynamic Analyzer (PDA) testing during pile installation on selected piles to confirm the design capacity.

The preliminary factored geotechnical resistances provided above will have to be re-evaluated and modified, as necessary, during detail design in consideration of additional subsurface investigation at the foundation elements.



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6.5.3 Frost Protection

All pile caps should be provided with a minimum 1.5 m of soil cover for frost protection as per OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*), as measured vertically from and perpendicular to the face of the abutment slope to the edge of the underside of the pile cap.

If adequate soil cover cannot be provided for the pile cap, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.6 Drilled Shaft (Caisson) Foundations

6.6.1 Founding Elevations

Drilled shafts (caissons) founded within the compact to dense silt to sandy silt may be considered for support of the abutments and piers for the proposed replacement structure. The following drilled shaft founding elevations may be used for preliminary design purposes:

Foundation Element	Proposed Underside of Pile Cap (m)	Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation
West Abutment	233 m	210 m	Very Stiff to Hard Silty Clay
Pier 1	227.5 m	210 m	Very Stiff to Hard Silty Clay
Pier 2	228.5 m	210 m	Very Stiff to Hard Silty Clay
Pier 3	228.5 m	210 m	Compact to Dense Silt
East Abutment	234 m	210 m	Compact to Dense Silt

If drilled shaft foundations are adopted, a temporary liner should be utilized to support the overburden soils during construction to minimize disturbance to the side walls and to control base disturbance/basal heave due to groundwater pressure/seepage. In addition, placement of concrete by tremie methods would be required.

6.6.2 Geotechnical Axial Resistance/Reaction

The following factored ultimate and serviceability geotechnical axial resistances (at ULS and SLS for 25 mm of settlement, respectively) may be used for design of drilled shaft (caisson) foundations:

Foundation Element	Drilled Shaft Diameter	Factored Ultimate Geotechnical Axial Resistance (at ULS)	Factored Serviceability Geotechnical Resistance (at SLS) for 25 mm of Settlement ¹
West Abutment, Pier 1 and Pier 2	0.9 m	1750 kN	N/A
	1.2 m	2900 kN	N/A
East Abutment and Pier 3	0.9 m	2000 kN	N/A
	1.2 m	3000 kN	N/A

Note: 1. The factored serviceability geotechnical resistance (at SLS) for 25 mm of settlement will be greater than the factored ultimate geotechnical axial resistance (at ULS) and as such, the SLS condition does not apply.



The preliminary factored geotechnical resistances provided above will need to be re-evaluated and modified, as necessary, during detail design in consideration of any additional subsurface investigation at the foundation elements.

6.6.3 Frost Protection

All caisson/pile caps should be provided with a minimum 1.5 m of soil cover for frost protection as per OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*), as measured vertically and perpendicular from the face of the abutment slope to the edge of the underside of the pile cap.

If adequate soil cover cannot be provided for the pile cap, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.7 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stem walls, and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill material, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of the abutment walls and associated retaining walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II, should be used as backfill behind the walls. Longitudinal drains and weep holes through the abutment stem walls should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement) and OPSD 3190.100 (Walls, Retaining and Abutment, Wall Drain).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC (2014)* Section 6.12.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.5 m behind the back of the wall (in accordance with Figure C6.20(a) of the *Commentary to the CHBDC 2014*). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (in accordance with Figure C6.20(b) of the *Commentary to the CHBDC 2014*). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:



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Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular 'A'	22 kN/m ³	0.43	0.27
Granular 'B' Type II	21 kN/m ³	0.43	0.27

Where the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design. Where the wall support allows lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the wall structure(s). The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the Commentary to the *CHBDC (2014)*.

6.8 Approach Embankments

6.8.1 Subgrade Preparation and Embankment Construction

Based on the existing topographic information, the existing Dunlop Street embankment side slopes are inclined at about 2 horizontal to 1 vertical (2H:1V). For the proposed widening of the Highway 400 embankments, the new side slopes should also be constructed at a Maximum inclination no steeper than 2H:1V. Where widening of the existing embankment occurs, benching the existing embankment side slopes should be carried out in accordance with OPSD 208.010 (Benching of Earth Slopes) to integrate the new fill into the existing slope fill.

It is understood that a 2.3 m to 3.3 m grade raise of the existing/widened portion of the approach embankments is proposed on the west and east side of Highway 400, resulting in new embankments about 7.5 m to 8.5 m high, respectively. As indicated on OPSD 202.010 (Slope Flattening), a minimum 2 m wide bench should be incorporated into the earth fill approach embankment slopes where the slopes are equal to or greater than 8 m high, such that the uninterrupted slope height does not exceed 8 m, as is the case at the east abutment.

To reduce the potential for erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod should be carried out as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS.PROV 804 (Seed and Cover).

6.8.2 Embankment Stability and Factored Settlement

Limit equilibrium slope stability analyses for the embankment was carried out using the commercially available program Slide (version 6.0), developed by Rocscience Inc., employing the Morgenstern Price method of analysis. For all analyses, the Factors of Safety (FoS) of numerous potential failure surfaces were computed for the critical embankment cross-section in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analysis, the FoS is equal to the inverse of the product of the consequence factor, Ψ , and the geotechnical resistance factor, ϕ_{gu} (i.e. $FoS = 1/(\Psi \cdot \phi_{gu})$). Accordingly, a target minimum FoS of 1.67 has been used for the design of the embankment slopes for the permanent conditions, as per Table 6.2 of CHBDC (2014). The stability analyses assume that all organics and other deleterious materials are removed prior to constructing the approach embankments. Based on the results of the analysis for deep-seated global failure surfaces, the FoS for the widening of the embankment is equal to or greater than 1.7 and as such, stability issues are not anticipated within the limits of the approach embankment widening.



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Settlement analyses were carried out using the commercially available program Settle3D (version 3.0), developed by Rocscience Inc.

The factored settlement associated with the up to about 2.3 m grade raise at the west approach embankment is estimated to be about 125 mm. This settlement is estimated to be comprised of about 25 mm of factored immediate settlement due to the compression of the native non-cohesive deposits and about 60 mm of factored primary settlement within the native cohesive deposit. The immediate settlement is expected to occur during and shortly after construction. Based on the average coefficient of consolidation (c_v) of about $3.9 \times 10^{-2} \text{ cm}^2/\text{s}$ estimated for the native cohesive deposit and the imposed loading conditions, and assuming two-way drainage of the approximately 11.3 m thick native cohesive deposit, it is estimated that 90 per cent of the factored primary consolidation settlement will be completed in about 80 days.

The factored settlement associated with the up to about 3.3 m grade raise at the east approach embankment is estimated to be about 205 mm. This settlement is estimated to be comprised of about 190 mm of factored immediate settlement due to the compression of the native non-cohesive deposits and about 15 mm of factored primary settlement within the native cohesive deposit. The immediate settlement is expected to occur during and shortly after construction. Based on the average coefficient of consolidation (c_v) of about $3.9 \times 10^{-2} \text{ cm}^2/\text{s}$ estimated for the native cohesive deposit and the imposed loading conditions, and assuming two-way drainage of the approximately 3.2 m thick native cohesive deposit, it is estimated that about 90 per cent of the factored primary consolidation settlement will be completed in about 5 days.

6.9 Construction Considerations

The following sections identify future construction considerations that may impact the detail design and construction of the new underpass structure.

6.9.1 Open-Cut Excavations

The construction of new spread/strip footings and/or pile caps will require excavations to depths of up to about 4 m below the existing Highway 400 grade and will be made through the existing embankment fill, native firm to very stiff clayey silt to silty clay, as well as the loose to dense sand deposit. The existing fill material is classified as a Type 3 soil and the native firm to very stiff clayey silt and loose to compact sand deposits are classified as a Type 3 soil above the water table and Type 4 below the water table, according to the Occupational Health and Safety Act (OHSA). As such, temporary open-cut excavations should be made with side slopes no steeper than 1H:1V above the groundwater table and with side slopes no steeper than 3H:1V below the groundwater table.

All excavations must be carried out in accordance with Ontario Regulation 213 (Ontario Occupational Health and Safety Act for Construction Projects) (as amended).

6.9.2 Temporary Protection Systems

Temporary protection systems may be required to facilitate the removal of the existing bridge foundations and construction of the abutments and piers. Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection System), and the lateral movement should meet Performance Level 2 provided that any existing adjacent utilities can tolerate this magnitude of



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deformation. The temporary protection system should be removed after completion of construction or cut-off at a depth of not less than 1.5 m below final ground surface.

The selection and design of the protection system will be the responsibility of the Contractor.

6.9.3 Control of Groundwater

Based on measured groundwater levels of about Elevations 229.7 m and 229.9 m, the bottom of excavations (i.e. the founding level) for spread/strip footings at the abutment and pier locations, and pile caps for deep foundation at the pier locations will likely require groundwater control measures to allow for construction in dry conditions. Depending on the groundwater level at the time of construction and the foundation alternative chosen for the bridge replacement, the excavation could extend between about 0.4 m and 2.4 m below the groundwater level.

The soils at the base of the excavations consist of water-bearing native loose sand, as well as firm to very stiff clayey silt. At this preliminary stage, it is anticipated that an active dewatering system (beyond pumping from sumps within the excavation) will be required to lower the groundwater level. It is recommended that the groundwater level be lowered to not less than 0.5 m below the footing/pile cap founding level. An accurate estimate of the groundwater pumping volumes cannot be made at the preliminary design stage.

It is anticipated that the zone of influence for dewatering operations would be relatively localized at the structure site. Assuming the dewatering system is properly constructed and operated such that there is no loss of fine soil particles, the dewatering operations are not expected to cause detrimental settlement in the clayey silt and sand deposits. However, the potential for settlement impacts on the structure foundations and any adjacent utilities should be assessed at the detail design stage.

6.9.4 Control of Ground and Groundwater for Drilled Shaft (Caisson) Construction

As noted in Section 6.6.1, running or flowing soil from the native non-cohesive deposits could occur during or after drilling the drilled shafts (caissons) and heave could occur at the caisson base as a result of groundwater pressure/seepage. If drilled shafts foundations are adopted, temporary liners should be used to support the overburden soils. Balancing groundwater pressures during construction by utilizing a head of water or bentonite drilling slurry inside the temporary liner will be required, and should be assessed at the detail design stage. In addition, placement of concrete by tremie methods would be required.

6.9.5 Obstructions

As noted in Section 3.2, the NQ casing used to advance Borehole DL1-1 sheared during extraction operations of the drill string and attempts to recover the 24.4 m of NQ casing were unsuccessful. Although Borehole DL1-1 was advanced away from the foundation footprint of the east pier and abutment, the abandoned NQ casing could potentially affect the construction deep foundations, should Dunlop Street be realigned further to the North. As such, it is recommended that the presence of the NQ casing be taken into consideration during detail design.



6.10 Recommendations for Future Work During Detail Design

Given the variability of the strata encountered in the boreholes in the previous and current foundation investigations, it is recommended that, during detail design additional site investigation and field testing be carried out at/within the footprint of each of the abutment and pier foundations and laboratory testing of soil samples be carried out for characterization. Boreholes should be extended to a sufficient depth to penetrate further into the very dense silty sand and silt and sand deposits to confirm the material characteristics (strength, composition) for the purpose of designing driven H-Pile foundations. Such a foundation investigation will allow for a more specific assessment of the subsurface conditions at these locations and for design of deep foundations, both to a lower tip elevation and to achieve higher factored geotechnical resistances if required.

In addition, although the lower silty clay deposit encountered in Borehole B11-1 has been classified as cohesive in the 2001 investigation, Atterberg limit testing was not carried out to confirm such classification. As such, it is recommended that sufficient laboratory testing be carried on samples of the native deposits to allow for the confirmation of the soil classification of the deposits, which will aid in the assessment of the soil strength and behaviour during detail design.

7.0 CLOSURE

This report was prepared by Ms. Madison Kennedy and Ms. Amelia Jewison, B.A.Sc., members of the geotechnical engineering group, and was reviewed by Mr. Christopher Ng, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., a Senior Consultant with Golder and Designated MTO Foundations Contact, conducted an independent quality control review of this report.

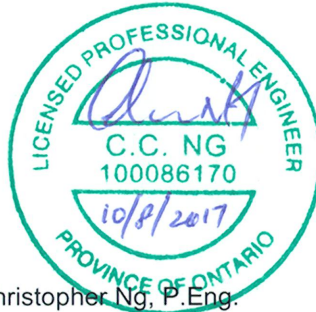


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Report Signature Page

GOLDER ASSOCIATES LTD.

Madison Kennedy, B.A.Sc.
Geotechnical Engineering Group



Christopher Ng, P. Eng.
Senior Geotechnical Engineer, Associate



Jorge M. A. Costa, P. Eng.
Designated MTO Foundations Contact, Senior Consultant

AJ/MCK/CN/JMAC/mck

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Commercial Software:

Slide (Version 6.0) by Rocscience Inc.

Settle^{3D} (Version 3.0) by Rocscience Inc

Ministry of Transportation Ontario:

Drawing SS103-11 Pile Driving Control

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 202.010	Slope Flattening
OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain



TABLES



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TABLE 1 – COMPARISON OF REPLACEMENT STRUCTURE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Spread/strip footings	<ul style="list-style-type: none"> Feasible for the support of new abutments and piers. 	<ul style="list-style-type: none"> Conventional excavation and construction techniques. Lower cost compared to deep foundations 	<ul style="list-style-type: none"> Require large footing excavation and disposal of a larger volume of soil compared to the excavation for a pile cap (excavation to about 4.5 m depth). Do not allow for integral abutment construction. Likely requires temporary protection system to allow for excavation/footing construction at centre pier. May require alternate footing design due to low factored serviceability geotechnical resistance available; footings could be constructed on a compacted granular pad thus achieving slightly greater geotechnical resistances. 	<ul style="list-style-type: none"> Estimated cost is approximately \$600/m³ for construction of shallow foundations. 	<ul style="list-style-type: none"> Footing subgrade must be protected from frost penetration
Steel H-piles or pipe piles	<ul style="list-style-type: none"> Feasible for the support of new abutments and piers with pile cap “perched” within the approach embankments or below the Highway 400 grade. 	<ul style="list-style-type: none"> Conventional construction methods for H-pile or steel pipe pile foundations. Abutment pile caps could be maintained higher than spread footings, thus reducing or eliminating 	<ul style="list-style-type: none"> Pipe piles not readily accepted for integral abutment construction; allow for semi-integral abutment configuration. 	<ul style="list-style-type: none"> Estimated cost is approximately \$250/m length for pile installation and \$600/m³ for pile cap construction. 	<ul style="list-style-type: none"> Minor potential for pile damage/deflection if cobbles and boulders are encountered during pile driving. Slightly greater risk in this regard for pipe piles as compared with



PRELIMINARY FOUNDATION AND DESIGN REPORT - HIGHWAY 400 DUNLOP STREET UNDERPASS

TABLE 1 – COMPARISON OF REPLACEMENT STRUCTURE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
		<ul style="list-style-type: none"> the depth of excavation and protection system requirements. Steel H-piles allow for integral abutment configuration. 			H-piles if boulders are encountered during pile driving.
Drilled Shaft (Caissons)	<ul style="list-style-type: none"> Feasible but not recommended for the support of abutments and piers 	<ul style="list-style-type: none"> Abutment pile caps could be constructed at the underside of the bridge or maintained higher than spread footings, or H-pile caps, reducing depth of excavation and protection system requirements, or caps can be constructed at level of underside of structure. Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles. 	<ul style="list-style-type: none"> Temporary liners will be required, plus special measures such as tremie placement of concrete; likely not possible to inspect caisson base. Precludes use of integral abutments. More expensive compared to shallow foundations. 	<ul style="list-style-type: none"> Estimated cost is approximately \$1,000/m length for caisson installation and \$600/m³ for pile cap construction; the cost may be higher to account for the use of a temporary liner. 	<ul style="list-style-type: none"> Risk of loosening and leaving in place disturbing founding soils at base of caissons.



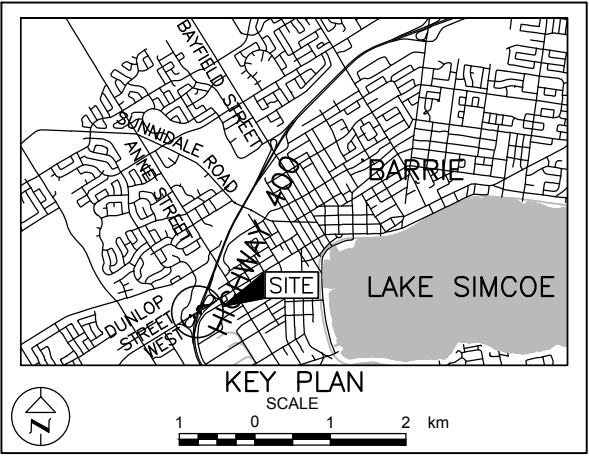
DRAWINGS

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

CONT No.
GWP No. 06-20016

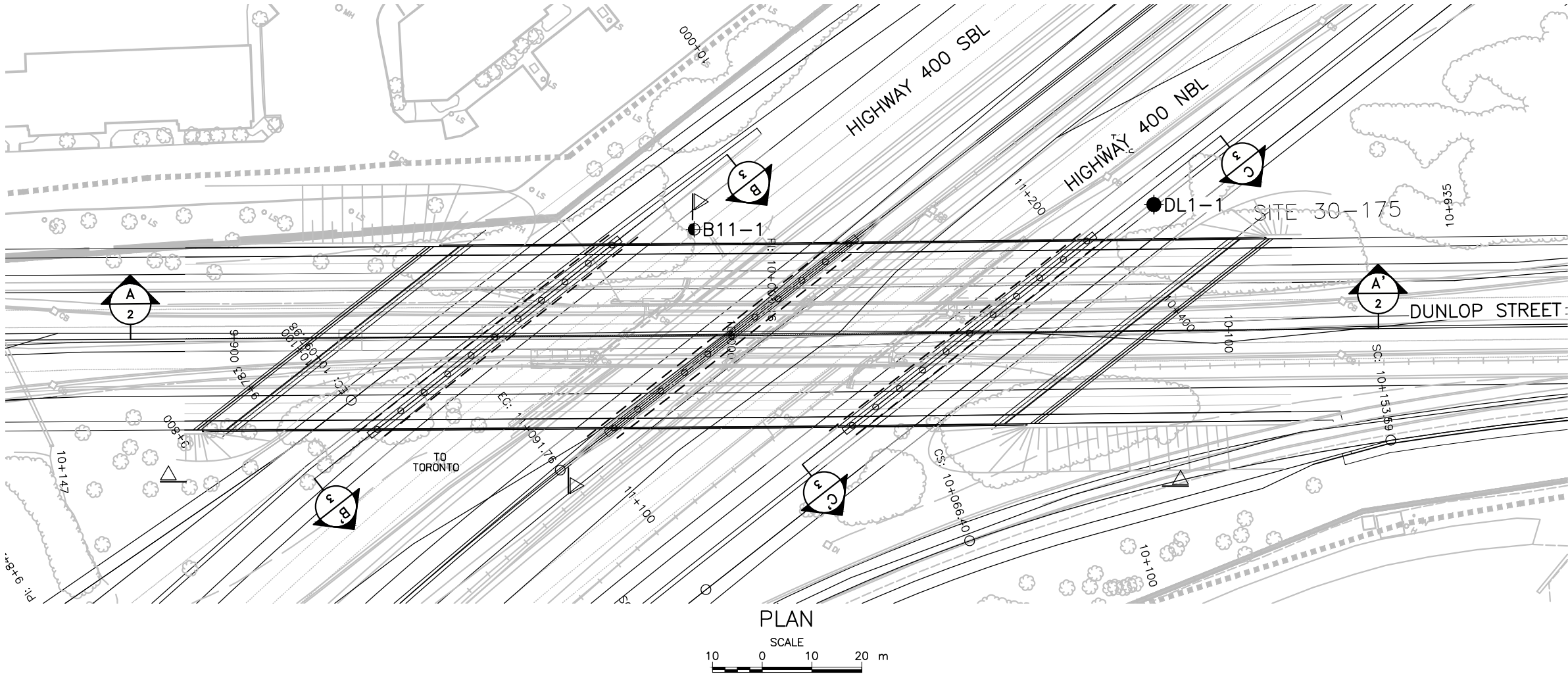
DUNLOP STREET UNDERPASS
HIGHWAY 400 WIDENING
BOREHOLE LOCATIONS


SHEET



LEGEND	
	Borehole - Current Investigation
	Borehole - Previous Investigation

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B11-1	231.2	4915592.6	287961.2
DL1-1	231.6	4915644.7	288037.8



NOTES

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

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

REFERENCE

General arrangement, designs, base plans, profile and surface data provided in digital format by AECOM, drawing file nos. "01_Dunlop Street Underpass_GA(2).dwg", received June 23, 2016, "X-Base_All.dwg", received January 27, 2016, "X-design_4th Line_Interim.dwg", received June 22, 2015, and "X-Surfaces.dwg", received April 14, 2015.



NO.	DATE	BY	REVISION
Geocres No. 31D-679			
HWY. 400	PROJECT NO. 14-1111-0002		DIST. .
SUBM'D. AJ	CHKD. MCK	DATE: 7/11/2017	SITE: 30-175
DRAWN: TB	CHKD. CN	APPD. JMAC	DWG. 1

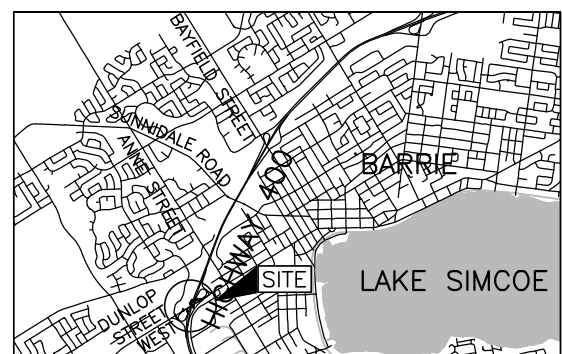
CONT No.
GWP No. 06-20016

DUNLOP STREET UNDERPASS

HIGHWAY 400 WIDENING







SOIL STRATA

SHEET



KEY PLAN
SCALE
1 0 1 2 km

LEGEND

-  Borehole – Current Investigation
 Borehole – Previous Investigation
 Seal
 Piezometer
 Standard Penetration Test Value
16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
 WL in piezometer, measured on MAR 15, 2001 and
MAR 15, 2017

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B11-1	231.2	4915592.6	287961.2
DL1-1	231.6	4915644.7	288037.8

NOTES

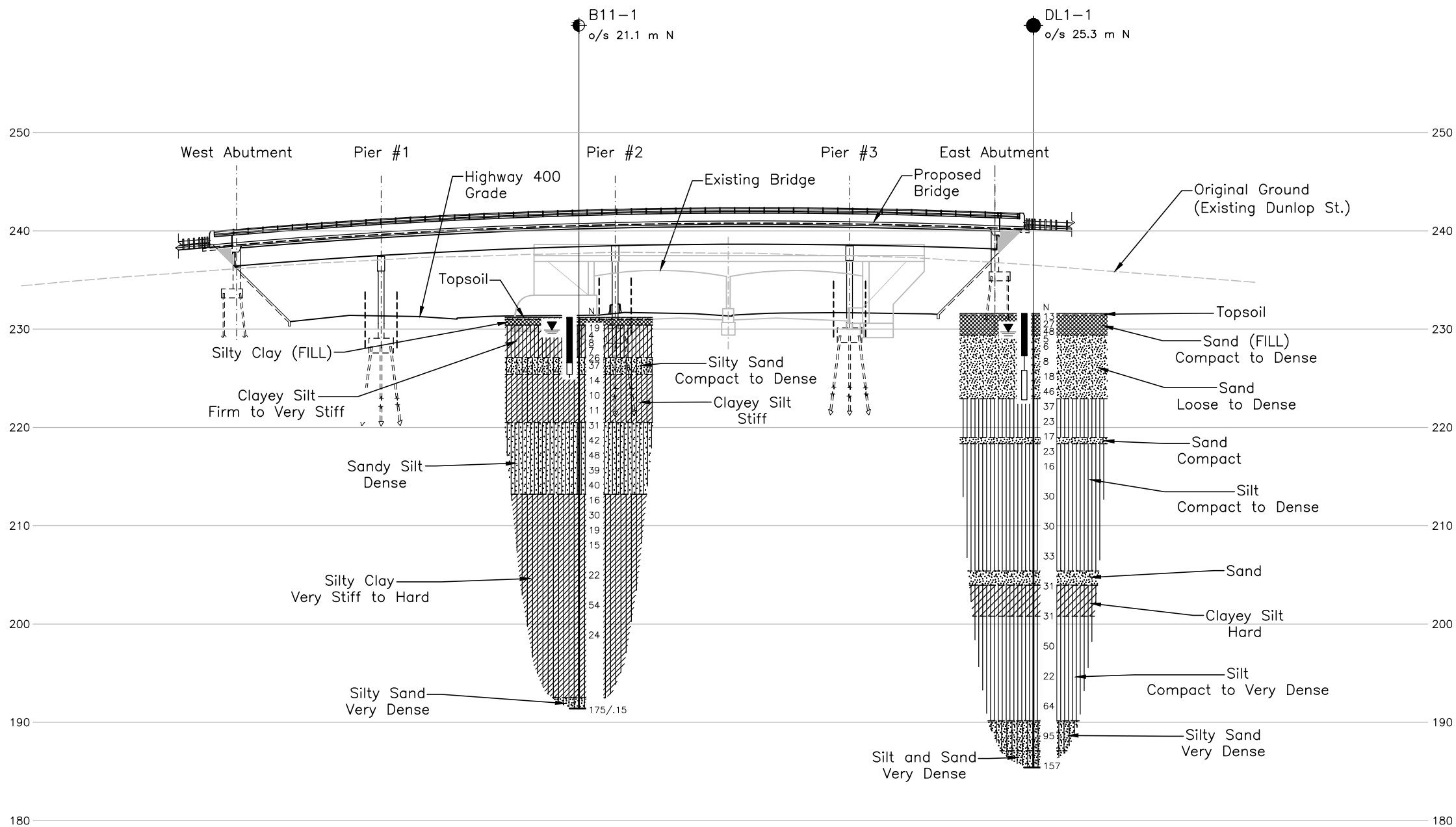
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Geocres No. 31D-679					
HWY. 400			PROJECT NO. 14-1111-0002		DIST. .
SUBM'D. AJ		CHKD. MCK	DATE: 7/11/2017		SITE: 30-175
DRAWN: TB		CHKD. CN	APPD. JMAC		DWG. 2



HIGHWAY 400 CENTERLINE PROFILE



Figure 1 shows two scales. The top scale is labeled 'HORIZONTAL SCALE' and has markings at 10, 0, 10, and 20 m. The bottom scale is labeled 'VERTICAL SCALE' and has markings at 5, 0, 5, and 10 m.



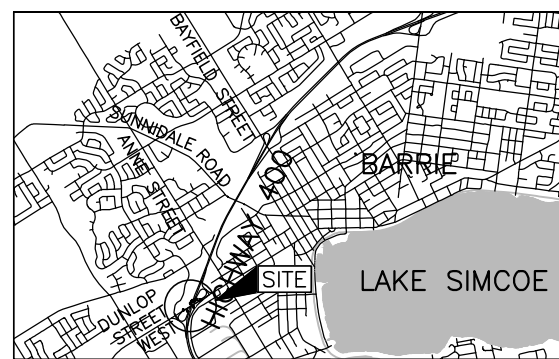
CONT No.
GWP No. 06-20016

DUNLOP STREET UNDERPASS

HIGHWAY 400 WIDENING







SOIL STRATA

SHEET



KEY PLAN
SCALE
1 0 1 2 km

LEGEND

-  Borehole – Current Investigation
 Borehole – Previous Investigation
 Seal
 Piezometer
 Standard Penetration Test Value
16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
 WL in piezometer, measured on MAR 15, 2001, and MAR 15, 2017

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B11-1	231.2	4915592.6	287961.2
DL1-1	231.6	4915644.7	288037.8

NOTES

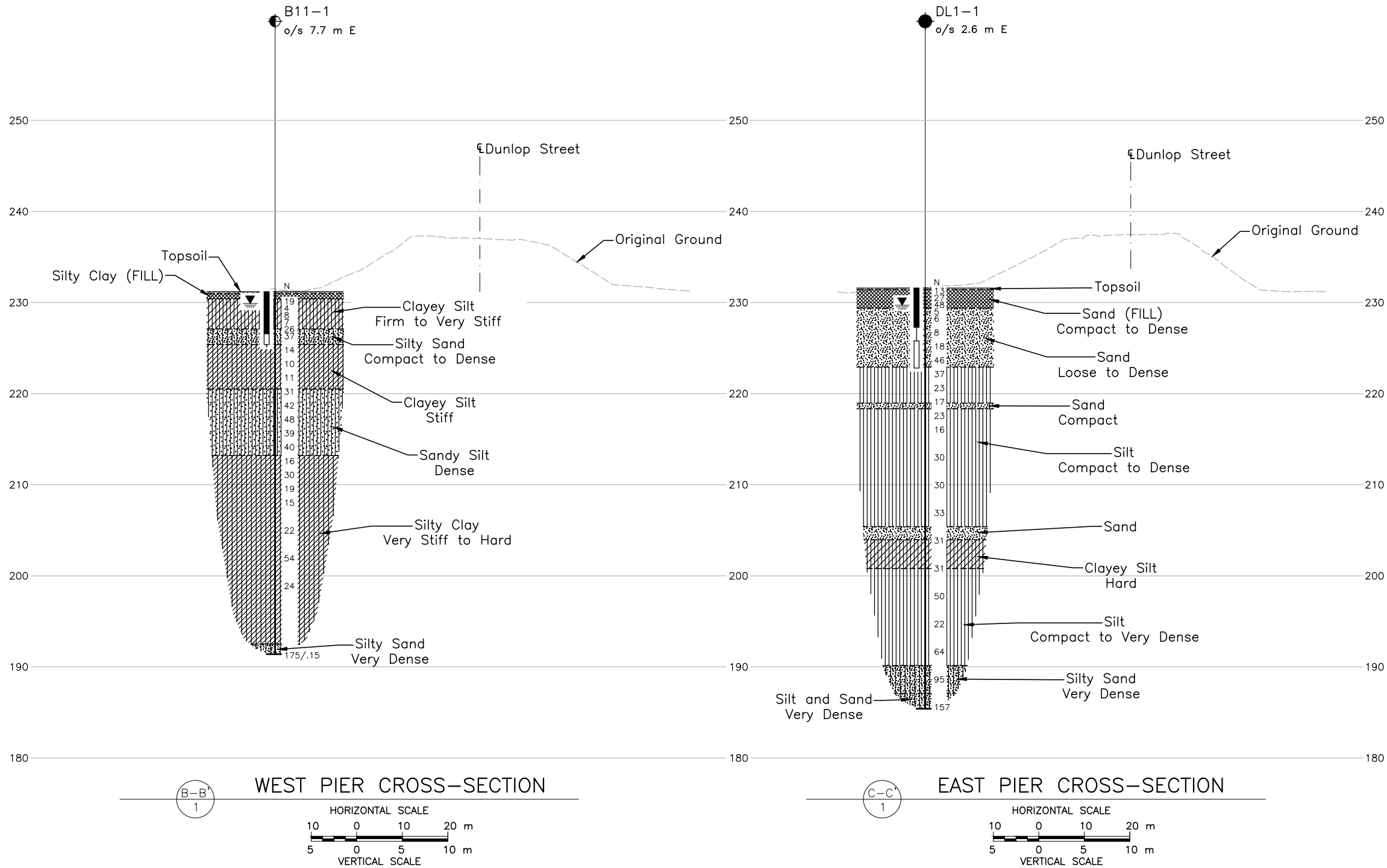
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-	-	-	-	-
NO.	DATE	BY	REVISION	
Geocres No. 31D-679				
HWY. 400		PROJECT NO. 14-1111-0002		DIST. .
SUBM'D. AJ	CHKD. MCK	DATE: 7/11/2017		SITE: 30-175
DRAWN: TB	CHKD. CN	APPD. JMAC		DWG. 3





APPENDIX A

Record of Borehole and Laboratory Test Results – Previous Investigation

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

(b) Cohesive Soils

Consistency

	c_u, s_u	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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (L.V.-laboratory vane test)
γ	unit weight

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

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LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

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

(c) 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

(d) Consolidation (one-dimensional)

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

(e) Shear Strength

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

Notes: 1. $\tau = c' + \sigma' \tan \phi'$
2. Shear strength = (Compressive strength)/2

PROJECT 001-1143F				RECORD OF BOREHOLE No B11-1				1 OF 3		METRIC		
W.P. 30-95-00				LOCATION N 4915592.6; E 287961.2				ORIGINATED BY SB/PKS				
DIST SW HWY 400				BOREHOLE TYPE SEE NOTE 1				COMPILED BY LCC				
DATUM Geodetic				DATE Jan.17-26/2001				CHECKED BY ASP				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)		
231.2	GROUND SURFACE											
0.0	Topsoil											
0.2	Silty Clay, trace sand (Fill) Brown											
230.4												
0.8	Clayey Silt to Silty Clay, trace sand Firm to very stiff Moist Grey		1	SS	19							
			2	SS	4							
			3	SS	8							
			4	SS	7							
227.1												
4.1	Silty Sand, trace clay and gravel Compact to dense Moist Grey		5	SS	26							
			6	SS	37							
225.4												
5.8	Silty Clay, trace sand Stiff Moist Grey Contains a layer of silty sand from 6.6m to 7m depth (Elev.224.6m to 224.2m)		7	SS	14							
			8	SS	10							
			9	SS	11							
220.5												
10.7	Sandy Silt containing silty clay interlayers Dense Wet Grey		10	SS	31							
			11	SS	42							
			12	SS	48							

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Continued Next Page

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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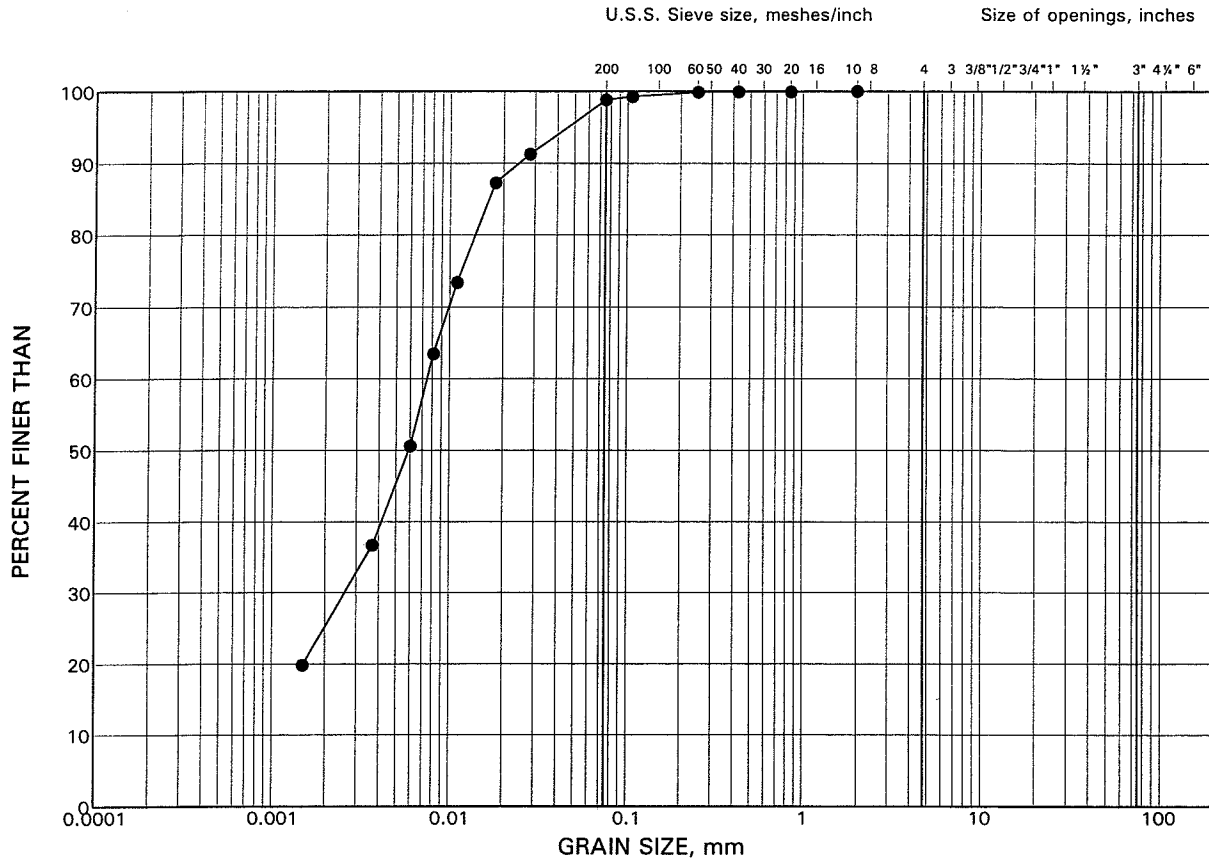
PROJECT 001-1143F			RECORD OF BOREHOLE No B11-1			3 OF 3			METRIC								
W.P. 30-95-00			LOCATION N 4915592.6; E 287961.2			ORIGINATED BY SB/PKS											
DIST SW HWY 400			BOREHOLE TYPE SEE NOTE 1			COMPILED BY LCC											
DATUM Geodetic			DATE Jan.17-26/2001			CHECKED BY ASP											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION		
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED			W _p W W _L 10 20 30			γ	GR SA SI CL		
192.5	Silty Clay, trace sand Very stiff Moist Grey		21	SS	24		201										
192							200										
191.4	Silty Sand Very dense Brown Wet		22	SS	175/15		199										
39.8	END OF BOREHOLE						198										
	Notes: 1. Hollow stem augers used to advance to 10.7m depth. After sampling, bentonite seal was placed between 11.3m and 8.8m depth, and augers were withdrawn. "N" casing was installed to 10.7m depth, then "B" casing was used for the remainder of the borehole. 2. During drilling operations, water level in open borehole was typically between 6.2m and 8.8m depth (Elev.225.0m to 222.4m). 3. Piezometer installed in second borehole drilled 4m west. Water level in piezometer measured at 1.3m depth (Elev.229.9m) on March 15, 2001.						197										
							196										
							195										
							194										
							193										
							192										

ON MOT 0011143F.GPJ ON MOT.GDT 14/1/02

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay

FIGURE 1



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE

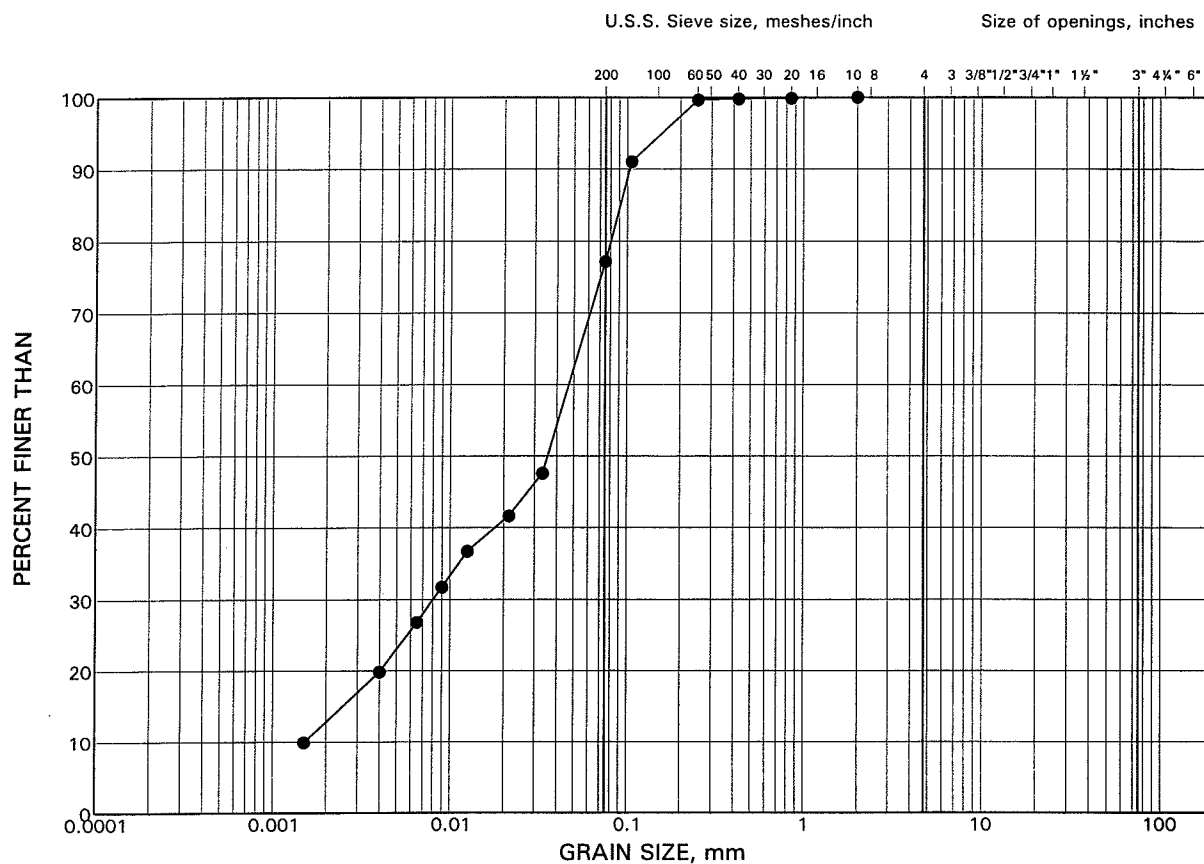
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B11-1	4	227.8

GRAIN SIZE DISTRIBUTION

Sandy Silt with Clayey Silt Interlayers

FIGURE 2



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B11-1	12	217.2



APPENDIX B

Record of Borehole and Laboratory Test Results – Current Investigation



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

* Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_{α}	secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
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
2

$\tau = c' + \sigma' \tan \phi'$
shear strength = (compressive strength)/2



LIST OF ABBREVIATIONS

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I. SAMPLE TYPE

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SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

III. SOIL DESCRIPTION

(a) Non-Cohesive Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

(b) Cohesive Soils

Consistency	Cu, Su	psf
Very soft	0 to 12 kPa	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

Dynamic Cone Penetration Resistance; Nd:

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

w	water content
w _p	plastic limit
w _l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

PROJECT 14-1111-0002

G.W.P.	06-20016	LOCATION	N 4915644.7; E 288037.8 MTM ZONE 10 (LAT. 44.380615; LONG. -79.710359)	ORIGINATED BY	ML
DIST	Central	HWY	400	BOREHOLE TYPE	Diedrich D-90, 108 mm I.D. Hollow Stem Augers, Tricone, NW Casing (Auto Hammer)
DATUM	Geodetic	DATE	November 27 to 30, 2016	COMPILED BY	SMD
				CHECKED BY	MCK

[illegible]

Continued Next Page

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

GTA-MTO 001 S:\CLIENTS\SMTO\HWY 400 BARRIE\02 DATA\GINT\1411110002 LAT LONG.GPJ GAL-GTA.GDT 2/8/17

PROJECT <u>14-1111-0002</u>		RECORD OF BOREHOLE No DL1-1		SHEET 2 OF 4		METRIC	
G.W.P. <u>06-20016</u>		LOCATION <u>N 4915644.7; E 288037.8 MTM ZONE 10 (LAT. 44.380615; LONG. -79.710359)</u>		ORIGINATED BY <u>ML</u>			
DIST <u>Central</u> HWY <u>400</u>		BOREHOLE TYPE <u>Diedrich D-90, 108 mm I.D. Hollow Stem Augers, Tricone, NW Casing (Auto Hammer)</u>		COMPILED BY <u>SMD</u>			
DATUM <u>Geodetic</u>		DATE <u>November 27 to 30, 2016</u>		CHECKED BY <u>MCK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								20 40 60 80 100	W _p	W	W _L										
--- CONTINUED FROM PREVIOUS PAGE ---																					
	SILT, some to trace clay, trace sand, with silty sand layers Compact to dense Grey Wet		13	SS	16		216														
							215														
							214														
			14	SS	30		213					11	11	10		0	1	85 14			
							212														
							211														
			15	SS	30		210						10		NP	0	1	95 4			
							209														
							208														
			16	SS	33		207														
							206														
205.4							205														
26.2	SAND Grey Wet						204						11	11	10						
204.0			17A				204						11	11	10						
27.6	CLAYEY SILT, sand seams Hard Grey Wet		17B	SS	31		203														
							202														

Continued Next Page


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

GTA-MTO 001 S:\CLIENTS\MTOWHY_400_BARRIER02_DATA\GINT\1411110002_LAT_LONG.GPJ GAL-GTA.GDT 2/8/17



GTA-MTO 001 S:\CLIENTS\MTO\HWY_400\BARRIE02\DATA\GINT\1411110002 LAT LONG.GPJ GAL-GTA.GDT 2/8/17

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

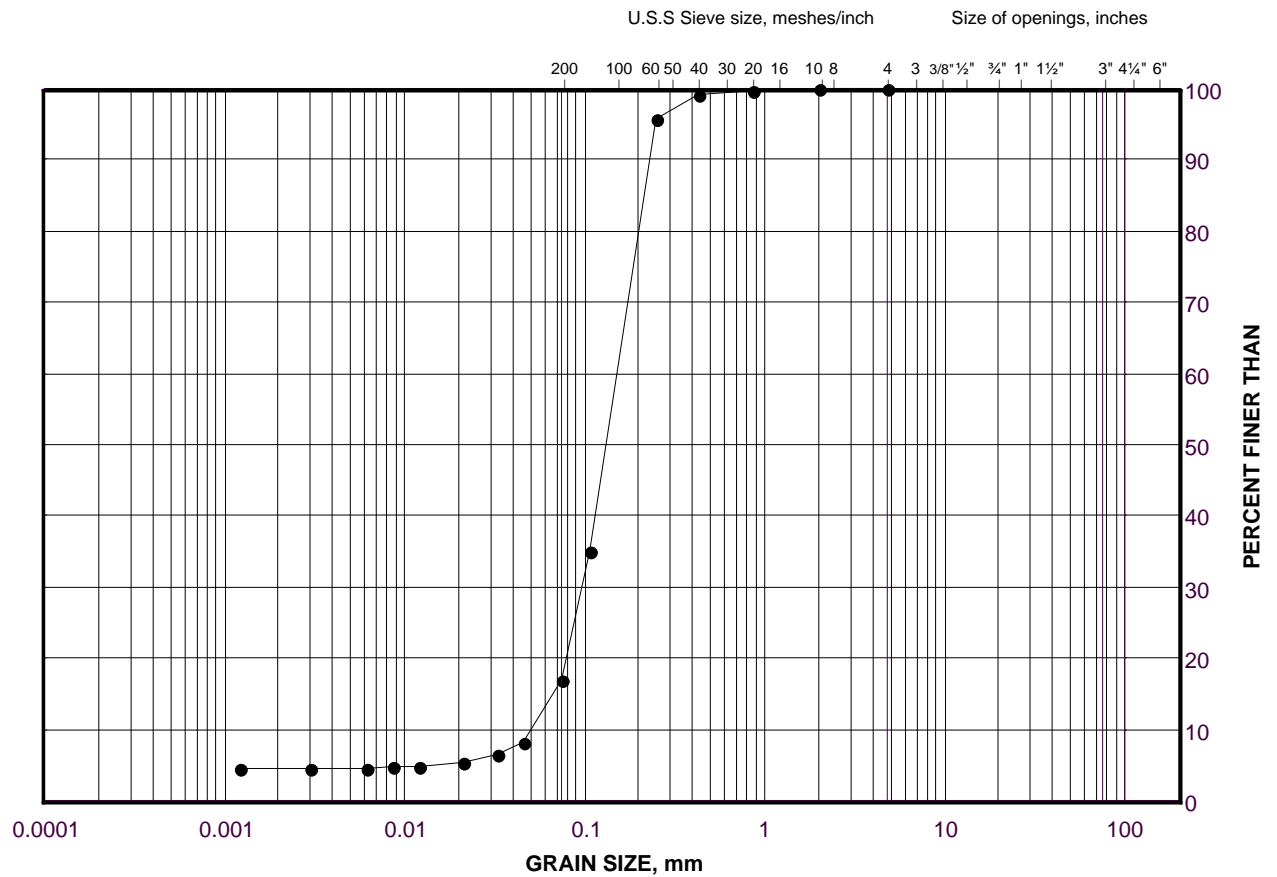
PROJECT		14-1111-0002		RECORD OF BOREHOLE No DL1-1		SHEET 4 OF 4		METRIC													
G.W.P.		06-20016		LOCATION		N 4915644.7; E 288037.8 MTM ZONE 10 (LAT. 44.380615; LONG. -79.710359)		ORIGINATED BY ML													
DIST		Central HWY 400		BOREHOLE TYPE		Diedrich D-90, 108 mm I.D. Hollow Stem Augers, Tricone, NW Casing (Auto Hammer)		COMPILED BY SMD													
DATUM		Geodetic		DATE		November 27 to 30, 2016		CHECKED BY MCK													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 10 20 30			kN/m ³					
185.4	SILT and SAND, trace clay Very dense Grey Wet		23	SS	157		186						○						0 55 44 1		
46.2	END OF BOREHOLE																				
184.2	END OF DCPT																				
47.4	NOTE: 1. Water levels measured in piezometer Date Depth (m) Elev (m) 15/03/17 1.9 229.7 2. Approximately 24.4 m of NQ casing sheared in the borehole between depths 21.8 m (Elev. 209.8 m) and 46.2 m (Elev. 185.4 m). Borehole and casing backfilled with bentonite pieces. An additional borehole was advanced about 1.7 m north to install the monitoring well.																				

GTA-MTO 001 S:\CLIENTS\MTOWHY_400_BARRIE\02_DATA\GINT\1411110002_LAT_LONG.GPJ GAL-GTA.GDT 2/8/17

GRAIN SIZE DISTRIBUTION

Sand

FIGURE B1



SILT AND CLAY SIZES			FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED			SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	DL 1-1	6	226.7

Project Number: 14-1111-0002

Checked By: ARJ

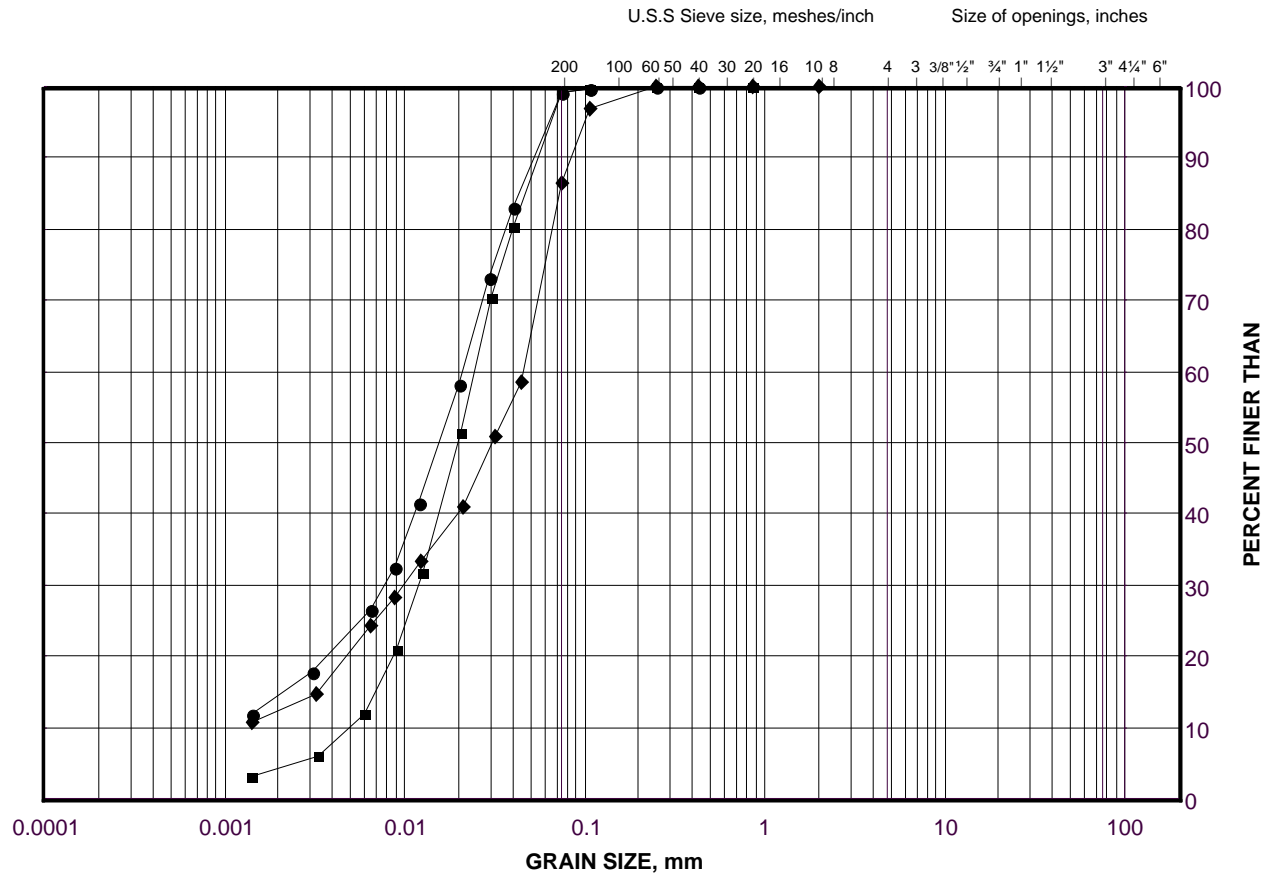
Golder Associates

Date: 24-Mar-17

GRAIN SIZE DISTRIBUTION

Silt (Upper)

FIGURE B2



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

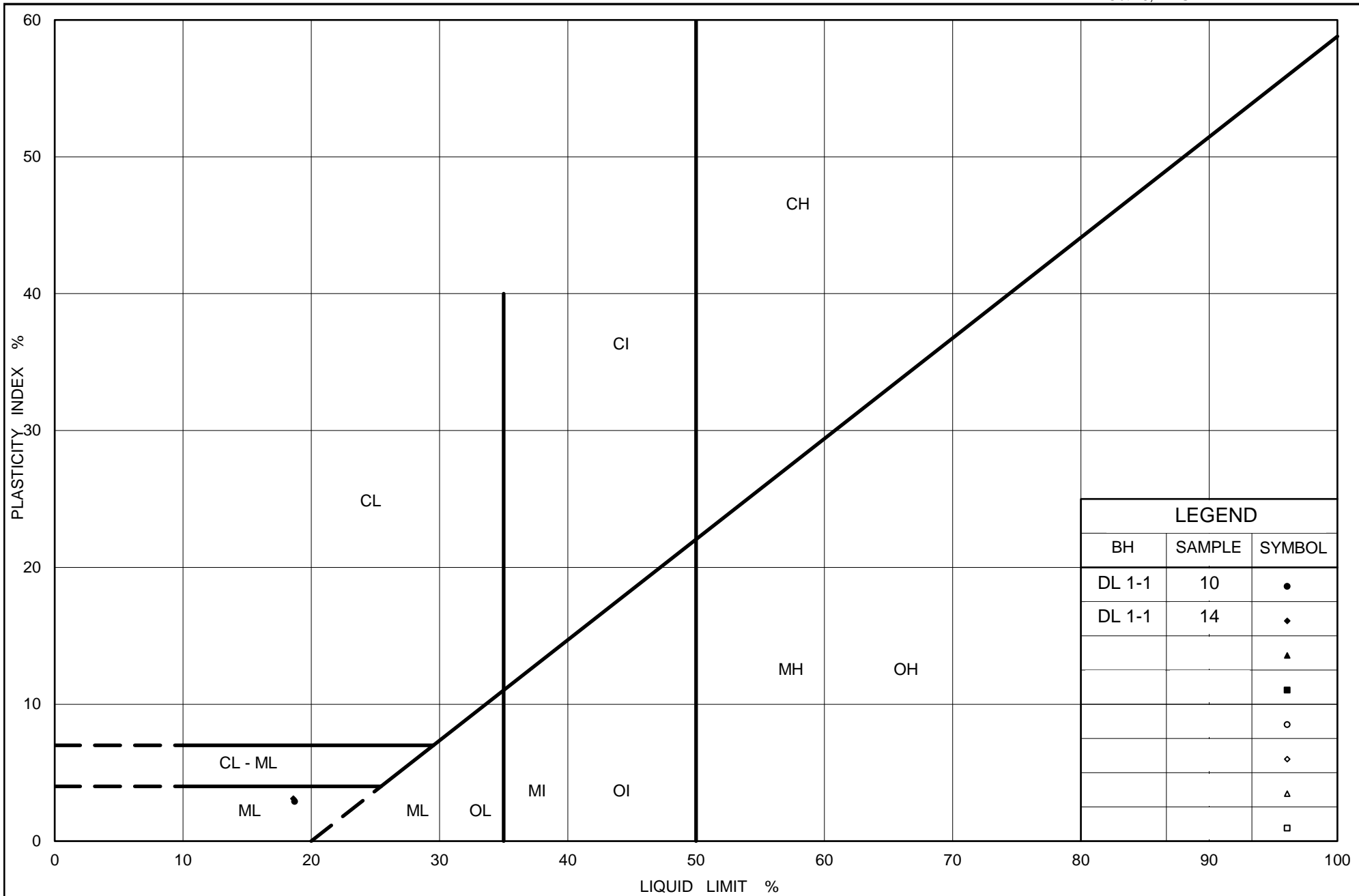
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	DL 1-1	14	213.0
■	DL 1-1	15	210.0
◆	DL 1-1	9	222.1

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Golder Associates

Date: 24-Mar-17



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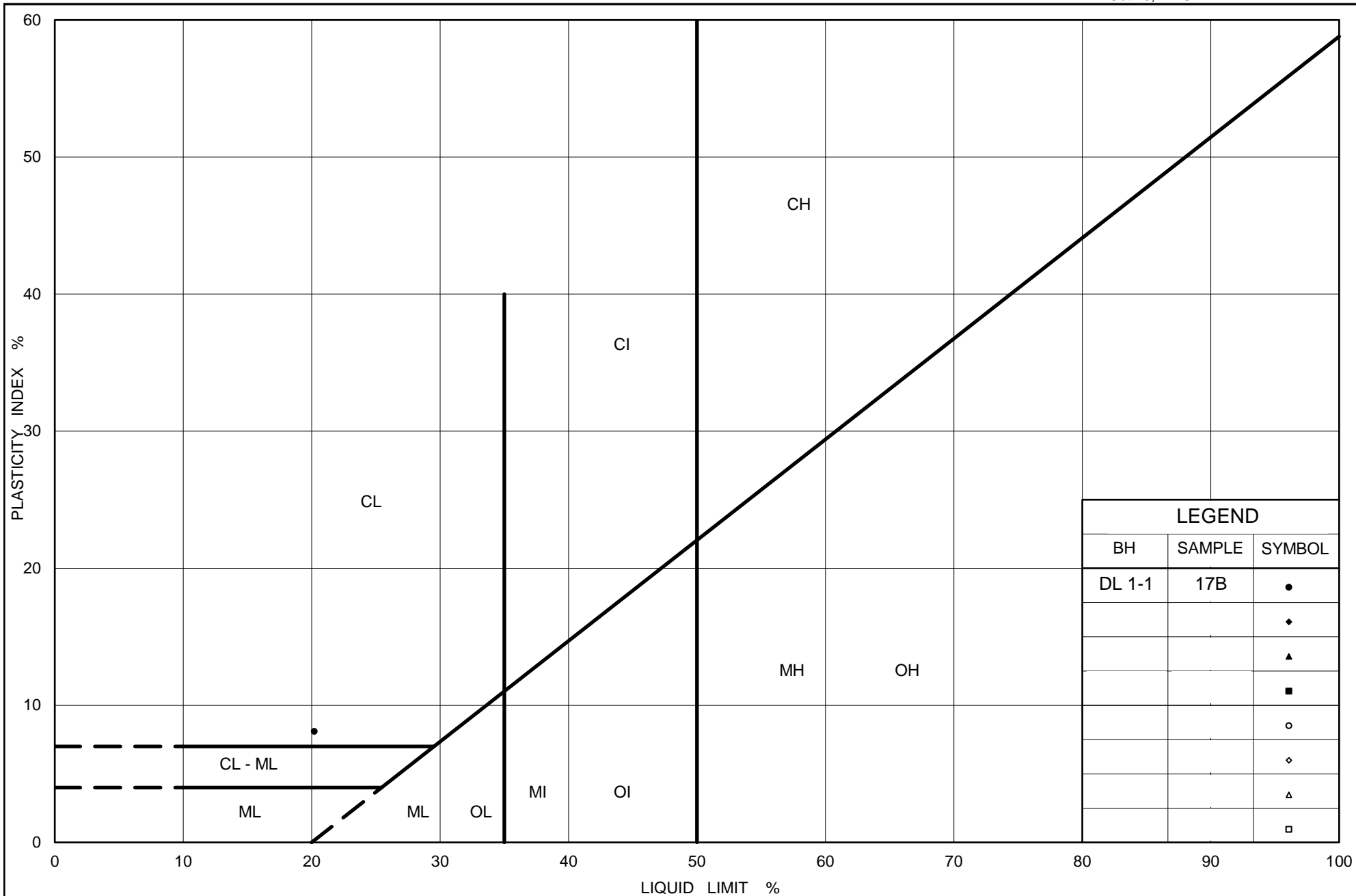
Ontario

PLASTICITY CHART Silt (Upper)

Figure No. B3

Project No. 14-1111-0002

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PLASTICITY CHART Clayey Silt

Figure No. B4

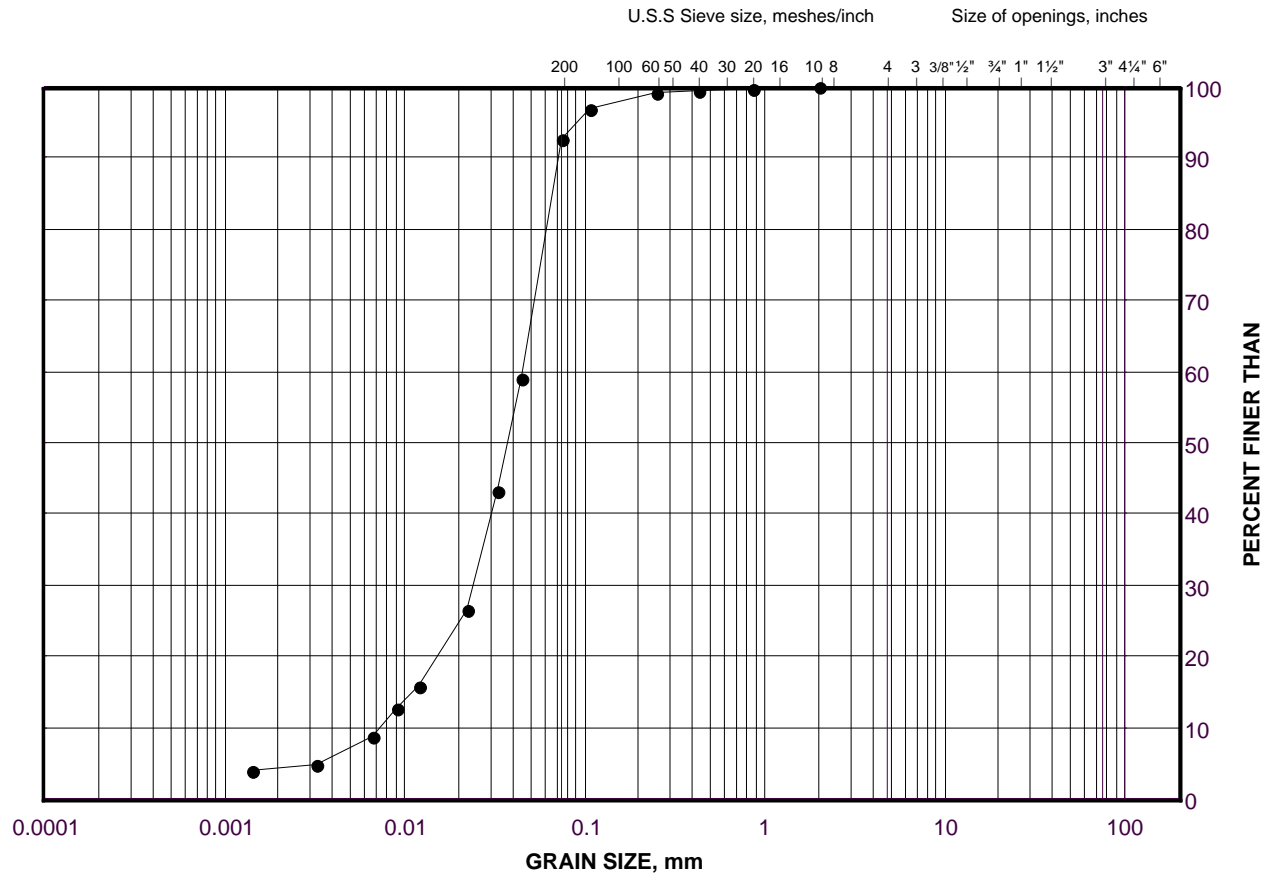
Project No. 14-1111-0002

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GRAIN SIZE DISTRIBUTION

Silt (Lower)

FIGURE B5



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

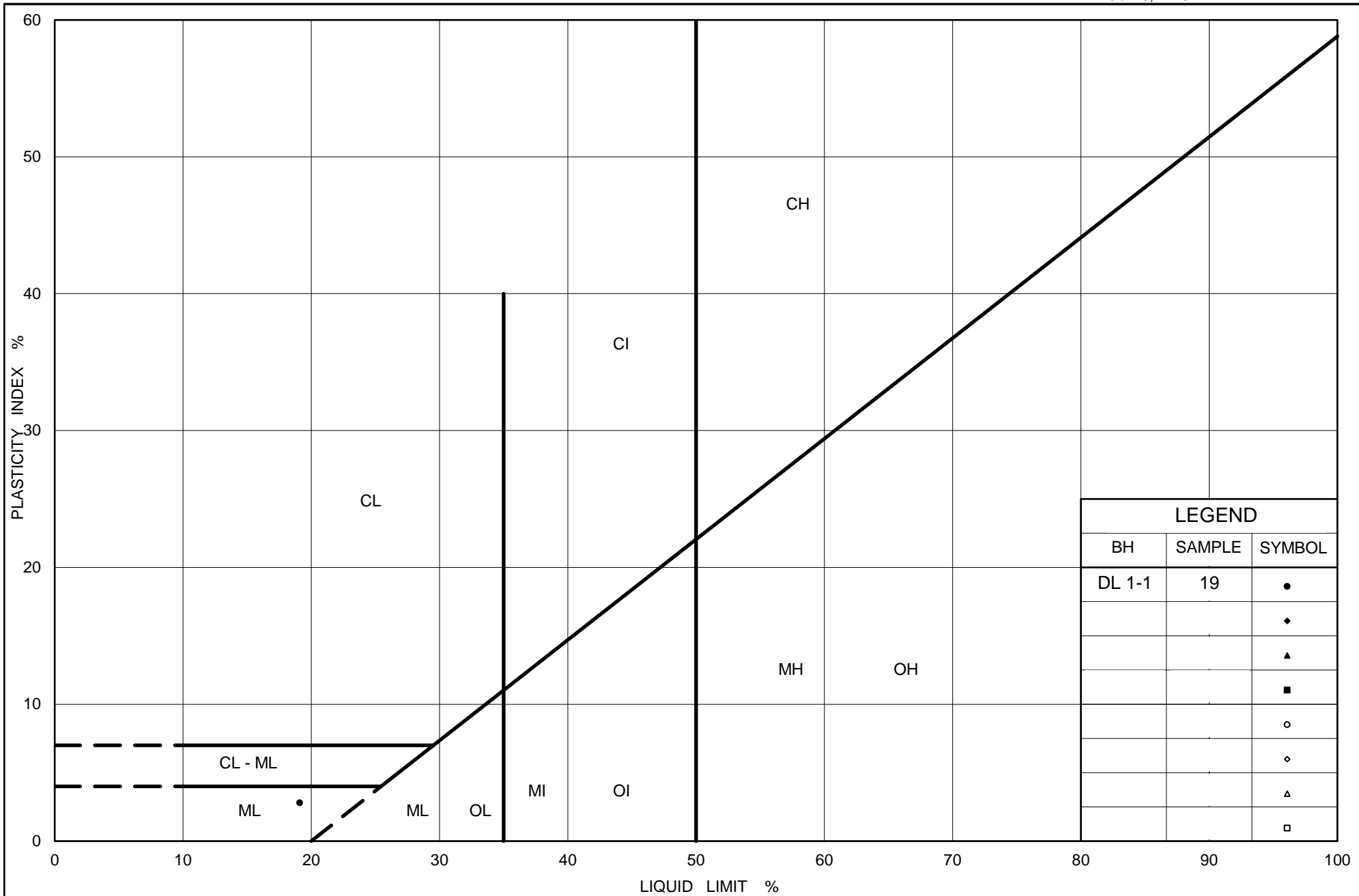
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	DL 1-1	21	191.6

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PLASTICITY CHART Silt (Lower)

Figure No.B6

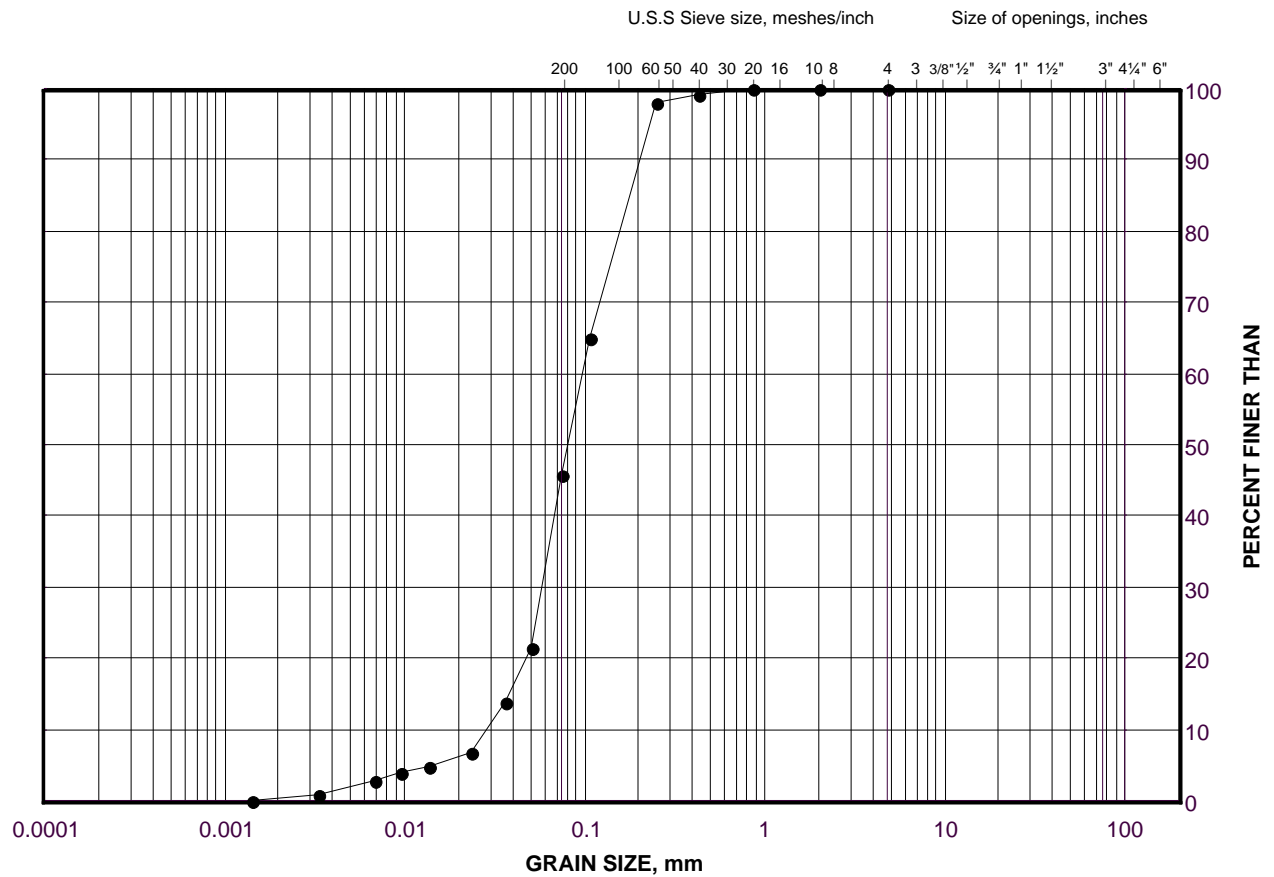
Project No. 14-1111-0002

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GRAIN SIZE DISTRIBUTION

Silt and Sand

FIGURE B7



SILT AND CLAY SIZES			FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED			SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	DL 1-1	23	185.6

Project Number: 14-1111-0002

Checked By: ARJ

Golder Associates

Date: 24-Mar-17

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South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario, L5N 7K2
Canada
T: +1 (905) 567 4444

