# Foundation Investigation and Design Report – Pond Mills Road Overpass Replacement- Highway 401 Rehabilitation from Wellington Road to Highbury Avenue, Design-Build Project

Highway 401 City of London, ON
West Region

DB Contract Number: 2022-3004
GWP 3032-11-00

Geocres No. 40I14-204

Prepared for:
CRH Canada Group Inc.

Prepared by:
Stantec Consulting Ltd.
300 – 675 Cochrane Drive
Markham, ON L3R 0B8

Project No. 165001239

February 02, 2023

# Table of Contents

## LIST OF TABLES

## LIST OF APPENDICES

FOUNDATION INVESTIGATION REPORT
For
GWP 3032-11-00
DB Contract Number 2022-3004

Pond Mills Road Overpass Replacement
Highway 401 Five Structure Replacements, Highbury Avenue Interchange Improvements, and Highway 401 Pavement Rehabilitation and Reconstruction
West Region
City of London, Ontario

# 1.0 INTRODUCTION

CRH Canada Group Inc. (CRH) is constructing the Highway 401 Five Structure Replacement project, which includes the Highbury Avenue Interchange improvements, and the Highway 401 rehabilitation and improvements in the City of London, on behalf of the Ontario for the Ministry of Transportation (MTO), under a Design-Build (DB) agreement. Stantec Consulting Ltd. (Stantec) was retained by CRH to undertake additional foundation investigations and detailed foundation designs for the project.

The project extends along Highway 401 from 675 m east of Wellington Road easterly 5.5 km to 630 m west of Old Victoria Road, along Pond Mill from 60 m north to 60 m south of Highway 401, and along Highbury Avenue from Bradley Avenue to Wilton Grove Road. The project includes following foundations engineering components:

- All deep cut areas and foundations for the new bridge structures, including:
  - CNR Overhead (London-Port Stanley Railway (Site No. 19X-0371/B1 & B2);
  - Pond Mills Overpass (Site No. 19X-0372/B1 & B2);
  - Highbury Avenue Underpass (Site No. 19X-0373/B0);
- Structural culvert replacements, including:
  - Tributary to Murray Drain Culvert (Site No. 19X-650/C0);
  - Elliot-Laidlaw Drain Culvert (Site No. 19X-651/C0);
- High mast lighting;
- Overhead signs;
- Retaining walls (at the bridges and overhead sign footings);
- 1.5:1 reinforced side slope between Station 25+110 and Station 25+270 westbound (changed to 2H:1V slopes); and
- Sewers and storm water management facility.

MTO reference numbers for this DB project are as follows:

GWP: 3032-11-00
DB Contract Number: 2022-3004

This foundation investigation report has been prepared specifically for the proposed Pond Mills Road overpass replacement (structure 19-372/B1 &B2) and the approach embankment widening. Other foundation engineering elements such as high mast light poles, median sewer and signs are reported under separate cover.

# 2.0 SITE DESCRIPTION

## 2.1 SITE LOCATION

The site location is shown on the Key Plan inset to Drawing No. 1 included in Appendix A.

## 2.2 GENERAL SITE DESCRIPTION

The Highway 401 Pond Mills Road Overpass is located in the City of London, Ontario. The location of the project is shown on the Key Plan, Figure 1. Highway 401 runs approximately in the southwest-northeast direction at the site, while Pond Mills Road runs generally north-south. For the purposes of this report, Highway 401 and Pond Mills Road are assumed to be oriented in an east-west direction and a north-south direction, respectively.

Pond Mills Road has two lanes of traffic in each direction and Highway 401 is a six-lane (three lanes in each direction) divided highway. The area adjacent to the bridge mainly consists of open green field (only at northwest quadrant) and developed lands. It is understood that the existing structure will be demolished and replaced with a new structure built at the same location as the existing structure.

## 2.3 EXISTING BRIDGE

The existing bridge structure at the Pond Mills Road overpass was constructed in 1955 and consists of a 10.5 m long single span, concrete tee beam structure. The total structure width is 33.6 m. The bridge deck was widened by about 2.0 m on each side in 1989 to accommodate an additional third lane in each direction, without modifications to the foundation elements. As per the as-original drawing available in the SDR, the original bridge abutments and retaining walls were founded on 3.7 m wide spread footings at elevations 267.8 m and 267.6 m for the west and east abutments, respectively.

The grade separation at the interchange was achieved by a partial cut profile along Pond Mills Road and fill profile along Highway 401, resulting in the approach embankments have a maximum height in the order of 2.5 m to 6.0 m above the adjacent prevailing ground surface. The existing approach embankments are close to 2H:1V. The embankment side slopes are well vegetated, and no visible signs of embankment settlement and slope instability were noted during the site reconnaissance and investigation.

## 2.4 SITE GEOLOGY

The physiographic mapping indicates that the Pond Mills Road Overpass site lies within the physiographic region known as the Westminster Moraine and is situated on an undrumlinized till plain (Chapman and Putnam, 1984). The surficial material consists of Port Stanley silty clay till and clayey silt till, in places covered by thin patches of lacustrine silt based on the available Pleistocene Geology map of area (Dreimanis, 1963).

According to Geological Survey of Canada 1:250,000 Geology map of Toronto-Windsor area (Map 1263A), the rock formation in the area of the site is described as medium brown, microcrystalline limestone of the Dundee Formation of the Hamilton Group of Middle Devonian Age (Sanford, 1969).

# 3.0 REVIEW OF PREVIOUS INVESTIGATIONS

The following GEOCRES reports were provided as part of the DB RFP:

- GEOCRES No 40I14-157 Preliminary Foundation Investigation and Design Report- Pond Mills Road Overpass Replacement, Highway 401 Interchange Improvements/Structural Replacements, City of London, Ontario, GWP 3054-11-00 (dated June 2015, prepared by Golder Associates LTD.).
- GEOCRES No 40I14-111 Foundation Investigation Report- Pond Mills Road Overpass Widening, Highway 401, District 2, London, Ontario (dated March 1987, prepared by MTO Engineering Materials Office, Foundation Design Section).

The above-mentioned GEOCRES reports were reviewed as part of the bid phase design, as part of the additional foundation investigation program development, and for preparation of the current report.

MTO Foundation Design Section drilled eight boreholes and Golder advanced four boreholes. The Golder Associates and MTO Foundation Design Section investigation findings are incorporated in the borehole location plan and stratigraphic section drawings included in Appendix A of this report. For reference, copies of borehole records, borehole location plan, stratigraphic sections, and laboratory test results from Golder Associates and MTO Foundation Design Section reports are also included in Appendix B.

Review of the existing information from previous investigations indicates that the subsurface stratigraphy within the overpass area consists generally of fill and topsoil materials to about elevations 269.4 m and 266.6 m overlying layers of clayey silt till, silt, sandy silt, clayey silt, and sand. The groundwater level was measured at about elevation 266.5 m to 267.0 m in the installed piezometer and standpipe monitoring well between February 11 and June 5, 2013.

# 4.0 STANTEC INVESTIGATION PROCEDURES

## 4.1 FIELD INVESTIGATION

The additional foundation investigation for the design-build overpass replacement consisted of advancing a total of six boreholes at the site, identified as PM-01 to PM-06. The new boreholes and previously drilled boreholes are well-distributed within the site to capture sufficient subsurface and groundwater information to support the proposed overpass replacement design and construction. The locations of the boreholes are shown on the Borehole Locations and Soil Strata Drawing Nos. 1 and 2, in Appendix A.

Prior to carrying out the investigation, Stantec contacted the public utility authorities to clear the borehole locations of both private and public utilities. MTO locates were also obtained from the MTO West Region.

The field drilling program was carried out from July 13 to July 18, and August 19 to August 24, 2022. The boreholes were advanced using continuous flight hollow and solid stem augers. The mud rotary technique was used while advancing PM-03 below 3 m depth. Drilling was carried out with truck-mounted and track-mounted drill rigs, both equipped for soil sampling.

The subsurface stratigraphy encountered in each borehole was recorded in the field by an experienced Stantec field technician. The soil samples were recovered at regular 0.76 m intervals for the critical zone / shallow depth (typically to 3.8 m depth) and 1.5 m interval to termination depth of boreholes (except at borehole PM-02, where 3.0 m interval was used below 28 m depth to termination depth of the borehole). Soil sampling was carried out using a 51 mm (outside diameter) split-tube sampler by conducting Standard Penetration Tests (SPTs) in accordance with the procedures outlined in *ASTM D1586 Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils*. All soil samples recovered from the boreholes were placed in moisture-proof bags. All recovered SPT samples were returned to our Markham laboratory for detailed classification and testing.

In-situ shear vane (MTO N-vane) tests in accordance with ASTM D2573 Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils were attempted on cohesive soils, where applicable, to determine the undrained shear strengths of cohesive soils. A pocket penetrometer was also utilized to estimate the strength/consistency of clayey soil samples at the site.

Boreholes PM-03 and PM-05 were advanced beyond the end of drilling to depths of 39.9 m and 16.0 m by carrying out dynamic cone penetration tests (DCPT) until the penetration resistance achieved a penetration rate of at least 100 blows/0.3 m.

Groundwater was also observed in several open boreholes during and upon completion of drilling. Monitoring wells were installed in boreholes PM-03 and PM-06. After completion of drilling, boreholes were backfilled with a mix of bentonite and drill cuttings. Boreholes advanced on the paved area were sealed with cold patch asphalt.

## 4.2 INVESTIGATION HOLE LOCATION AND ELEVATION SURVEY

The borehole locations and respective ground surface elevations were surveyed by Stantec Geomatics personnel using Trimble R10-2 (horizontal accuracy of 8 mm+0.5 ppm and vertical accuracy of 15 mm+0.5 ppm as per the Trimble GNSS datasheet) to meet the survey accuracy requirements (vertical accuracy of 0.1 m and horizontal accuracy of 0.5 m) of the Guideline for MTO Foundation Engineering Services V2. Summary information pertaining to the Stantec boreholes included in this report is given in Table 4.1.

**Table 4.1: Borehole Information Summary**

| Investigation Borehole | MTM Zone 11 Coordinates | | Ground surface elevation (m) | Total depth drilled or advanced (m) | End of borehole elevation (m) | Number of soil samples |
|---|---|---|---|---|---|---|
| | Northing | Easting | | | | |
| PM-01 | 4755685.0 | 411524.8 | 275.9 | 18.9 | 257.0 | 15 |
| PM-02 | 4755695.2 | 411548.2 | 275.6 | 40.2 | 235.4 | 25 |
| PM-03 | 4755747.4 | 411584.7 | 270.6 | 38.7 | 231.9 | 28 |
| PM-04 | 4755725.5 | 411619.6 | 274.5 | 18.9 | 255.6 | 15 |
| PM-05 | 4755752.6 | 411593.6 | 271.1 | 14.3 | 256.8 | 12 |
| PM-06 | 4755649.5 | 411552.5 | 269.2 | 15.1 | 254.1 | 13 |

## 4.3 LABORATORY TESTING

All samples were taken to Stantec's Markham laboratories where they were subjected to a detailed visual and tactile examination. The geotechnical laboratory testing program completed on the borehole samples is summarized in Table 4.2. Two soil samples were tested for pH, soluble sulphate content, chloride content, and resistivity. Samples remaining after testing will be placed in storage for a period of one year after issuance of the final report. After the storage period, the samples will be discarded unless we are directed otherwise by MTO.

**Table 4.2: Geotechnical Laboratory Testing Program**

| Test Description | Number of Tests | Testing Firm |
|---|---|---|
| Moisture Content | 117 | By Stantec |
| Atterberg Limits | 21 | By Stantec |
| Grain Size Distribution (sieve & hydrometer) | 27 | By Stantec |

# 5.0 SUBSURFACE CONDITIONS

## 5.1 OVERVIEW

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are displayed on the Record of Borehole sheets contained in Appendix C. An explanation of the symbols and terms used to describe the Borehole Records is also provided in Appendix C. The results of geotechnical laboratory testing are also presented on Figures D1 to D11 contained in Appendix D.

Borehole location plans and stratigraphic cross-sections of the soils encountered within the boreholes along and across the proposed overpass are provided on Drawing Nos. 1 and 2 in Appendix A.

The stratigraphic boundaries on the borehole records and the strata plot are inferred from non-continuous sampling and therefore represent transitions between soil types rather than exact boundaries between geological units. The conditions will vary beyond the borehole locations. The stratigraphy generally consisted of:

- Near-surface asphalt, topsoil and/or fill materials (pavement, grading and embankment fills)
- Upper clayey silt to silty clay
- Upper silt to sandy silt
- Lower silty clay
- Lower silt
- Basal silty clay to clay

The groundwater level was measured at the installed monitoring wells at elevations 266.5 m and 266.4 m.

The subsurface conditions identified during the current investigation are in general agreement with the previous investigations’ findings (e.g. stratigraphy including soil composition and depositional structure).

Detailed descriptions of the subsurface and groundwater conditions found in the current investigation program are provided in the following sections.

## 5.2 OVERBURDEN

### 5.2.1 Ground Surface Cover

#### 5.2.1.1 Pavement

The boreholes drilled on the highway (BHs PM-01, PM-02, and PM-04) encountered approximately 280 mm of asphalt pavement.

#### 5.2.1.2 Topsoil

Boreholes PM-03, PM-05, and PM-06 were advanced in the highway embankment toe areas covered by grass and weeds adjacent to the highway. The surficial overburden materials were characterized as topsoil and ranged in thickness from 100 mm to 250 mm. The topsoil thickness may vary across the site and measured topsoil thickness at specific borehole locations should not be relied on for stripping quantity estimate.

### 5.2.2 Fills

Granular embankment and grading fill materials extended to 0.8 m to 2.3 m at boreholes PM-01, PM-02, PM-04, and PM-06:

- Granular fill materials, described as sand and gravel and silty sand, were encountered under the Highway asphalt pavement in BHs PM-01, PM-02, and PM-04 and were 2.0 m, 0.8 m, and 1.2 m thick. Standard Penetration Test (SPT) N-values measured within the granular fill materials ranged from 14 to 40 blows per 0.3 m, indicating a compact to dense relative density. The measured moisture content ranged from 3 to 8%.
- A 500 mm thick silty sand fill material was also encountered below the topsoil at borehole PM-06. This fill layer was in a very loose state, based on the measured N-value of 3 blows per 0.3 m, and extended to 0.8 m below the existing ground or to elevation of 268.4 m.

Cohesive fill layers were encountered in all borehole locations expect PM-06: Clayey silt, trace to some gravel and trace sand, was encountered below the granular fill in PM-01, PM-02, and PM-04 and below the topsoil at PM-05. A sandy clay to clayey silt with trace gravel was encountered below topsoil in PM-03. The cohesive fill material ranged in thickness from 2.1 m to 6.4 m, and extended to 2.2 m to 8.7 m below the existing ground or to elevations of 268.9 m to 267.2 m.

Based on the measured SPT N-values which ranged from 7 to 38 blows per 0.3 m penetration, the cohesive fill materials at the site have a firm to hard consistency, but generally a stiff to very stiff consistency (average SPT N-value of 15 blow per 0.3 m penetration). Measurement of undrained shear strength using MTO N-vane was attempted within the cohesive fill (e.g. at 3.1 m depth at PM-02, at 4.5 m and 5.5 m depth at PM-04) but encountered refusal implying an undrained shear strength higher than 100 KPa. Unconfined compressive strength, estimated by using the pocket penetrometer on recovered split-tube soil samples, were between 0.75 kgf/cm$^2$ to 4 kgf/cm$^2$ and suggested an undrained shear strength of 40 kPa to 210 kPa or a firm to hard consistency. Unconfined compressive strength of 4.5 kgf/cm$^2$ or more (undrained shear strength of 240 kPa or more) were also implied within the cohesive fill (on soil samples from 6.4 m and 7.9 m depths from borehole PM-01) from maximum pocket penetrometer readings. Index tests carried out on representative samples of the grading and embankment fill materials yielded the following results:

Granular fills:

- Gravel: 20%
- Sand: 59%
- Silt and Clay: 13%
- Moisture Content: 8%

Cohesive fills:

- Gravel: 2 to 13%
- Sand: 27 to 36%
- Silt: 29 to 33%
- Clay: 29 to 35%
- Moisture Content: 11 to 24%

Atterberg limit tests carried out on cohesive samples of the fill materials measured Liquid Limits of 23 to 30 percent, Plastic Limits of 11 to 17 percent and corresponding plasticity indices of 12 to 14. The Unified Soil Classification System (USCS) group symbol for the fill material is clayey silt and sandy clayey silt (CL).

The results of grain size distribution testing and the corresponding plasticity charts for samples of the fill materials are displayed on Figures D1 to D3 of Appendix D, respectively. Test results are also presented on the Records of Borehole Sheets included in Appendix C.

### 5.2.3 Upper Clayey Silt to Silty Clay

Below the embankment and grading fills, a clayey silt to silty clay deposit was encountered in all boreholes. The deposit thickness ranged from 1.6 m to 7.3 m and extended to depths ranging from 6.1 m to 14.9 m below ground surface (elevations 268.9 m to 260.7 m).

SPT N-values measured within this deposit ranged from 8 to 34 blows per 0.3 m (average 18 blows per 0.3 m) indicating a stiff to hard consistency, but generally stiff to very stiff. No undrained shear strength measurements were made using the MTO N-vane due to the inferred undrained shear strength being higher than 100 KPa (could not push the vane into soils or could not turn the vane). Unconfined compressive strength, estimated by using the pocket penetrometer on recovered split-tube soil samples, were between 1 kgf/cm$^2$ to 4.25 kgf/cm$^2$ and suggested an undrain shear strength of 50 kPa to 230 kPa or a stiff to hard consistency. Unconfined compressive strength of 4.5 kgf/cm$^2$ (undrained shear strength of 240 kPa) or more were also implied within the clayey silt to silty clay deposit (on soil sample from 1.8 m depth from borehole PM-06) from maximum pocket penetrometer readings.

Index tests carried out on representative samples from the surficial clayey silt till layer yielded the following results:

- Gravel: 0 to 6%
- Sand: 1 to 17%
- Silt: 34 to 66%
- Clay: 33 to 50%
- Moisture Content: 9 to 19%

Atterberg limit tests carried out on representative samples from this layer measured Liquid Limits of 19 to 32, Plastic Limits of 12 to 16 percent and corresponding plasticity indices of 7 to 18. The USCS group symbol for this layer is clayey silt (CL).

The results of grain size distribution testing and the corresponding plasticity charts for samples of the upper clayey silt till layer are displayed on Figures D4 and D5 of Appendix D, respectively.

### 5.2.4 Upper Silt to Sandy Silt

Boreholes PM-03, PM-04, PM-05, and PM-6 encountered a layer of silt to sandy silt under the clayey silt to silty clay. All above boreholes fully penetrated this layer and confirmed the layer is 5.2 m to 7.5 m thick, extending to elevations ranging from 257.0 m to 255.5 m.

SPT N-values measured within this deposit ranged from 0 to 53 blows per 0.3 m penetration (average SPT N-value of 23 blows per 0.3 m) suggesting the deposit is very loose to very dense, but generally loose to dense. A localized very loose silt with trace sand (based on measured N-values of nil) zone was encountered at PM-04 from 15.4 m to 17.5 m depth (or elevations 259.1 m to 257.0 m) and loose sandy silt to silt (based on measured N-values of 5 and 7) was encountered at PM-05 between 8.7 m and 11.2 m depth (or elevations 262.4 m to 259.9 m). The very loose to loose silt samples retrieved from boreholes PM-04 and PM05 showed sensitivity to disturbance (jelly-like and liverish appearance after disturbance) at their natural moisture contents.

Index tests carried out on a representative sample of the silt to sandy silt yielded the following results:

- Gravel: 0
- Sand: 6 to 29%
- Silt: 67 to 85%
- Clay: 4 to 22%
- Moisture Content: 13 to 21%

Atterberg limits test was attempted on one representative sample from this layer which resulted in a non-plastic outcome. The USCS group symbol for this layer is silt and silt with sand (ML).

A grain size distribution plot for a representative sample of this layer is displayed on Figure D6 in Appendix D.

### 5.2.5 Lower Clayey Silt

A layer of silty clay was encountered below the upper silty clayey silt to silty clay in boreholes PM-01 and below the silt to sandy silt deposit in boreholes PM-03, PM-04, PM-05, and PM-06. The silty clay layer thickness was 3.9 m and extended to a depth of 18.2 m below ground surface (corresponding elevation 252.4 m) in borehole PM-03, where it was fully penetrated. Boreholes PM-01, PM-04, PM-05, and PM-06 were terminated within this layer after 0.2 m to 4.1 m penetration.

SPT N-values measured within this layer ranged from 14 to 61 blows per 0.3 m (with an average of 33) suggesting the silt to silty clay has a stiff to hard consistency, but generally is very stiff to hard. Beyond the end of drilling of borehole PM-05, dynamic cone penetration tests (DCPT) were carried out to depth of 16.0 m where a penetration rate of at least 100 blows/0.3 m was achieved. Unconfined compressive strength, estimated by using the pocket penetrometer on recovered split-tube soil samples, were between 2 kgf/cm$^2$ to 4.25 kgf/cm$^2$ and suggested an undrained shear strength of 110 kPa to 230 kPa or a very stiff to hard consistency. Unconfined compressive strength of 4.5 kgf/cm$^2$ (undrained shear strength of 240 kPa) or more were also implied within the clayey silt deposit (on soil sample from 17.1 m depth from borehole PM-01) from maximum pocket penetrometer readings.

Index tests carried out on a representative sample from the surficial clayey silt to silty clay layer yielded the following results:

- Gravel: 0 %
- Sand: 0 to 2%
- Silt: 45 to 67%
- Clay: 32 to 55%
- Moisture Content: 13 to 20%

Atterberg limit tests carried out on a representative sample from this layer measured Liquid Limits of 19 to 28 percent, Plastic Limits of 12 to 13 percent and corresponding plasticity indices of 7 to 15. The USCS group symbol for this layer is clayey silt (CL).

The results of grain size distribution testing and the corresponding plasticity charts for samples of the clayey silt to silty clay are presented on Figures D7 and D8 of Appendix D, respectively.

### 5.2.6 Lower Silt

A lower silt layer was encountered below the silty clay to clayey silt layer in BH PM-02 at a depth of 13.7 m below ground surface (elevation 260.7m) and below the lower silty clay layer in BH PM-03 at 18.2m below ground surface (elevation 252.4 m). The deposit is 14.7 m and 3.7 m thick and extended to elevations of 246.0 m and 248.6 m, respectively. SPT N-values within this silt layer ranged from 20 to more than 100 blows per 0.3 m penetration, indicating a compact to very dense relative density, but generally dense to very dense.

Index tests carried out on a representative sample of the silt yielded the following results:

- Gravel: 0
- Sand: 1 to 20%
- Silt: 60 to 76%
- Clay: 20 to 23%
- Moisture Content: 14 to 24%

Atterberg limits tests were attempted on two representative samples from this layer which both resulted in a non-plastic outcome. The USCS group symbol for this layer is silt and silt with sand (ML).

A grain size distribution plot for a representative sample of this layer is displayed on Figure C9 in Appendix C.

### 5.2.7 Clayey Silt Till

A basal clayey silt till was encountered in Boreholes PM-02 and PM-03 at 29.6 m and 21.9 m below ground surface (elevations 246.0 m and 248.6 m, respectively). Both boreholes were terminated within that layer at 40.2 m and 38.7 m depths (elevations of 235.4 m and 231.9 m, respectively). The silty clay to clay deposit can be considered very stiff to hard, based on the measured SPT N-value of 21 to 51 blows per 0.3 m penetration (with an average of 34). Beyond the end of drilling of borehole PM-03, dynamic cone penetration tests (DCPT) were carried out to depth of 39.9 m where a penetration rate of at least 100 blows/0.3 m was achieved. Unconfined compressive strength, estimated by using the pocket penetrometer on recovered split-tube soil samples, were between 1 kgf/cm$^2$ to 3.5 kgf/cm$^2$ and suggested un undrained shear strength of 50 kPa to 190 kPa or a stiff to very stiff consistency.

Index tests carried out on a representative sample of the silt yielded the following results:

- Gravel: 1 to 2%
- Sand: 13 to 21%
- Silt: 41 to 47%
- Clay: 37 to 40%
- Moisture Content: 12 to 17% (with the exception of sample SS24 from PM-03 which had a moisture content of 52%)

Atterberg limit tests carried out on a representative sample from this layer measured Liquid Limits of 21 to 22 percent, Plastic Limits of 11 to 12 percent and corresponding plasticity index of 10. The USCS group symbol for this layer is clayey silt till (CL).

The results of grain size distribution testing and the corresponding plasticity charts for samples of the clayey silt till are presented on Figures C10 and C11 of Appendix C, respectively

### 5.2.8 Bedrock

Bedrock was not encountered to the termination depth of the boreholes.

### 5.2.9 Groundwater

Groundwater conditions are observed during drilling operations and upon drilling completion in open boreholes. Cave-in depths were also recorded. Two monitoring wells were installed in boreholes PM-03 and PM-06. The groundwater levels recorded in the boreholes are summarized in Table 5.1 below.

**Table 5.1: Measured and Inferred Groundwater Levels**

| Borehole No | Date | Groundwater Level (m) | | Remark |
|---|---|---|---|---|
| | | Depth | Elevation | |
| PM-01 | Upon completion | 10.9 | 265.0 | Caved-in at 18.0 m |
| PM-02 | Upon completion | 12.2 | 263.4 | Open |
| PM-03 | Upon completion | 4.6 | 266.0 | Open |
| | September 12-14, 2022 | 4.1 | 266.5 | Monitoring Well |
| PM-04 | Upon completion | 9.1 | 265.4 | Caved-in at 13.4 m |
| PM-05 | Upon completion | dry | - | Open |
| PM-06 | Upon completion | 4.6 | 264.6 | Open |
| | September 12-14, 2022 | 2.8 | 266.4 | Monitoring Well |

The recorded groundwater levels are generally consistent with measured groundwater levels of 266.5 m to 267.0 m reported by Golder in 2013. Fluctuations in the groundwater level due to seasonal variations or in response to a particular precipitation event should be anticipated.

## 5.3 CHEMICAL TESTING

The results of the chemical analysis on two samples of the site soils are provided in Table 5.2 below.

**Table 5.2: Results of Chemical Analysis**

| Borehole No | Sample No. | Depth (m) | pH | Chloride (µg/g) | Sulphate (µg/g) | Resistivity (Ohm-cm) |
|---|---|---|---|---|---|---|
| PM-02 | SS6 | 4.6 | 6.6 | 206 | 16 | 2120 |
| PM-03 | SS8 | 7.6 | 6.8 | 8 | 96 | 4520 |

# 6.0 MISCELLANEOUS

The field work was carried out under the supervision of Wuhib Tamrat, Akshat Shukla, Justin Moleta, and Binoy Debnath, under the direction of Gwangha Roh, Ph.D., P. Eng.

The drilling equipment was supplied and operated by Landshark Drilling based in Brantford and DBW Drilling Inc. based in North York, Ontario. Traffic control service was provided by CRH Group Inc. Chemical testing for pH, soluble sulphate, and chloride content, and resistivity was out by Agat Laboratories based in Mississauga.

The location and elevation survey of the investigation holes was carried out by Stantec's Geomatics Group based in London. Geotechnical laboratory testing was carried out at Stantec's Markham laboratory.

This report was prepared Ramin Ghassemi, Ph.D., P.Eng. and reviewed by Gwangha Roh Ph.D., P.Eng. and by Raymond Haché, M.Sc., P. Eng., Designated Principal MTO Foundation Contact.

# 7.0 CLOSURE

A subsurface investigation is a limited sampling of a site. The subsurface conditions described herein are based on information obtained at the specific investigation hole locations. Some variation in conditions between and beyond these locations must be anticipated. Should any conditions at the site be encountered which differ from those described for the investigation hole locations, we request that we be notified immediately to review the additional information and assess if revisions or changes to the content of this report are warranted.

Respectfully Submitted;

**STANTEC CONSULTING LTD.**

LICENSED PROFESSIONAL ENGINEER
R. GHASSEMI
100536678
2023/02/02
PROVINCE OF ONTARIO

Ramin Ghassemi

Ramin Ghassemi, Ph.D., P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
G. H. ROH
100148471
2023/02/02
PROVINCE OF ONTARIO

Gwangha Roh, Ph.D., P. Eng.
Senior Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
J.G.A.R. HACHÉ
17713504
2023/02/02
PROVINCE OF ONTARIO

Raymond Haché, M.Sc., P.Eng.
Senior Principal, Designated Principal MTO Foundation Contact

\\cd1215-f01\work_group\01216\active\1650\165001239\reports\reports\08-pond mills\final fidr\165001239_darft_hwy401highbury_fidr_pondmills_20230131.docx

FOUNDATION DESIGN REPORT
For
GWP 3032-11-00
DB Contract Number 2022-3004

Pond Mills Road Overpass Replacement
Highway 401 Five Structure Replacements, Highbury Avenue Interchange Improvements, and
Highway 401 Pavement Rehabilitation and Reconstruction
West Region
City of London, Ontario

# 8.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

## 8.1 PROJECT DESCRIPTION AND BACKGROUND

### 8.1.1 Project Purpose/Description

This project involves the replacement of five structures, Highbury Avenue Interchange improvement and Highway 401 pavement rehabilitation and improvement. As part of the project, the existing single span overpass carrying Highway 401 over Pond Mills Road will be replaced with a new two-span structure. The overpass replacement will also include the approach embankment widening and grade change. This report covers only the design and construction of Pond Mills Road overpass replacement, and other structures will be provided under separate covers.

### 8.1.2 Proposed Bridge Replacement

Based on the General Arrangement Drawing provided by Stantec Structural team, the proposed bridge will be constructed at a similar centreline alignment (with 20°30’ skew angle to the existing Highway 401 centreline) as the existing bridge. The new overpass will be single span integral abutment structure with a total length of 54.0 m (including approach slabs), and approximately 40.0 m from centreline to centreline of abutment walls. The overall width of the new bridge will be approximately 50.7 m, which will accommodate a future 3.75 m widening. The integral abutments will be supported on a single row of driven steel H-piles. The new overpass will be constructed in stages to accommodate removal of the existing bridge, and construction of the new. The existing approach embankments will be widened and raised to accommodate the new overpass width and profile. The existing bridge structure will be removed to footing elevation.

Key approximate elevations associated with the proposed new overpass are as follows:

| | |
|---|---|
| Existing Highway 401 grade at west-bound centreline | elevations 277.16 m to 276.58 m |
| Existing Highway 401 grade at east bound centreline | elevations 277.26 m to 276.71 m |
| Propose bridge west abutment bottom | elevation. 271.60 m |
| Proposed bridge east abutment bottom | elevation 271.25 m |
| Pond Mills Avenue grade at Highway 401 centreline | elevation 268.61 m |

### 8.1.3 Degree of Site Understanding and Consequence Classification

The Canadian Highway Bridge Design Code (CHBDC S6-19) requires an assessment of the "degree of site and prediction model understanding" as a component of the geotechnical engineering investigation and/or services. The site and prediction model understanding considers the geotechnical properties of the soils underlying the site and the accuracy and degree of confidence regarding the numerical performance prediction models to be used to estimate the geotechnical serviceability limit states reactions and ultimate limit states resistances.

Based on the scope of subsurface investigations completed and available subsurface information related to this site, a "Typical Understanding" has been adopted for foundation design assessment purposes. except that a "High" degree of understanding has been adopted for assessment of embankment stability where slip surfaces develop through imported/manufactured granular fill materials. MTO highway Standards Branch Provincial Memorandum #2020-01 (dated March 23, 2020) was also considered for the embankment global stability assessment if majority of instability is located within the proposed widening section which will be built using controlled materials (high degree of understanding).

The consequence classification has been assumed as "Typical Consequence" in accordance with Section 6.5 of the Commentary on CHBDC S6-19. Should the consequence classification change, the foundation assessment and recommendations provided below should be reviewed and revised accordingly.

## 8.2 GEOTECHNICAL DESIGN PARAMETERS

The soil conditions encountered at the overpass site generally consist of grading and embankment fill materials underlain by native soils consisting of upper deposits of clayey silt to silty clay and silt to sandy silt, underlain by lower silty clay and silt deposits, and a basal silty clay to clay deposit.

The results of the current investigation and previous investigations indicate that the subsurface conditions are generally consistent within the overpass area. Two geotechnical models (soil profiles), one for the the north half of the east abutment and another for the remaining of site, have been prepared for the overpass foundation design and embankment stability and settlement evaluation.

The soil profile is summarized on the following tables and on Drawings No. E1 and E2 in Appendix D. The geotechnical parameters identified in the soil profiles were developed based on a synthesis of the borehole data, the measured penetration resistance values, and laboratory index test results (including moisture contents) of soil samples obtained in the investigation

The elevations provided on the drawing and table reflect a synthesis of the borehole data; reference should be made to the Borehole Records for the range of conditions encountered.

### Table 8.1: Geotechnical Model for Highway 401 Pond Mills Road Overpass

| **Elevation (m)** | | **Soil Type** | **Design Soil Parameters** | | | |
|---|---|---|---|---|---|---|
| **From** | **To** | | **Total Unit Weight$^2$ $\gamma$ (kN/m$^3$)** | **Drained Friction Angle $\phi'^3$ (°)** | **Undrained Shear Strength $S_u^3$ (kPa)** | **E(MPa)$^4$** |
| Ground Surface | 268 | FILL: Firm to hard CLAYEY SILT to SILTY CLAY / very loose to dense SAND & gravel, silty sand, and silt. | 21.0 (cohesive fills)<br>22.0 (granular fills) | 30 (cohesive fills)<br>30 (granular fill)$^6$ | 75 (cohesive fills)<br>N/A (granular fill) | 20-30 (cohesive fills)<br>30-40 (granular fills) |
| 268 | 264 | Upper CLAYEY SILT to SILTY CLAY (stiff to hard) | 21.0 | 32 | 200 | 50 |
| 264 | 256 | Upper SILT to SANDY SILT (very loose to very dense)$^5$ | 21.0 | 30 | - | 10 |
| 256 | 252 | Lower CLAYEY SILT (stiff to hard) | 20.5 | 32 | 150 | 35 |
| 252 | 247 | Lower SILT (dense to very dense) | 22.5 | 33 | - | 40 |
| 247 | 229 | Basal SILTY CLAY to CLAY (very stiff to hard) | 21.0 | 32 | 250 | 50 |

Notes:

N/A Not Applicable

[1] A static groundwater level at elevations of 267 m is recommended for use in foundation design

[2] Submerged unit weight ($\gamma'$) should be used below the groundwater level.

[3] The friction angles are applicable to drained conditions only and the shear strengths are applicable to undrained conditions only

[4] Compressibility Parameters: E = Soil Modulus

[5] Borehole PM-01 is cohesive within this zone, however, as a synthesis of the site conditions the parameters listed are conservative for the conditions at that location.

[6] Based on the existing fill embankment performance.

**Table 8.2: Geotechnical Model for Highway 401 Pond Mills Road Overpass (the north half of the east abutment)**

| Elevation (m) | | Soil Type | Design Soil Parameters | | | |
|---|---|---|---|---|---|---|
| From | To | | Total Unit Weight[2] $\gamma$ (kN/m$^3$) | Drained Friction Angle $\phi'$[3] (°) | Undrained Shear Strength $S_u$[3] (kPa) | E(MPa)[4] |
| Ground Surface | 268 | FILL: Firm to stiff SANDY CLAY to CLAYEY SILT | 21.5 | 30 | 75 | 20-30 |
| 268 | 262 | Upper CLAYEY SILT to SILTY CLAY (stiff to hard) | 20.5 | 32 | 150 | 50 |
| 262 | 257 | Upper SILT to SANDY SILT (loose to dense) | 20.5 | 30 | - | 10 |
| 257 | 252 | Lower SILTY CLAY (stiff to hard) | 21.5 | 32 | 150 | 35 |
| 252 | 247 | Lower SILT (dense to very dense) | 22.0 | 33 | - | 40 |
| 247 | 229 | Basal SILTY CLAY to CLAY (very stiff to hard) | 21.5 | 32 | 200 | 50 |

Notes:

N/A Not Applicable

[1] A static groundwater level at elevations of 267 m is recommended for use in foundation design

[2] Submerged unit weight ($\gamma'$) should be used below the groundwater level.

[3] The friction angles are applicable to drained conditions only and the shear strengths are applicable to undrained conditions only

[4] Compressibility Parameters: E = Soil Modulus

## 8.3 FROST PENETRATION

In accordance with OPSD 3090.101, the design frost penetration depth for foundations, f, at the site is 1.2 m. Therefore, all foundation elements such as footings and pile caps should be provided with a minimum of 1.2 m of soil cover or equivalent insulation for protection against frost heaving.

This depth of frost penetration should also be considered in the design of frost tapers adjacent to the bridge abutment and retaining wall backfill zones.

## 8.4 SEISMIC DESIGN CONSIDERATIONS

### 8.4.1 Site Class

The seismic site class determination is based on the soil conditions in the upper 30 m of the stratigraphy as encountered in the boreholes for the Geotechnical Investigation.

Based on the current and previously done geotechnical investigations' findings, this site is assessed to be Seismic Site Class D as per CHBDC S6-19 Commentary Table 4.1.

### 8.4.2 Seismic Performance Category

As per the CHBDC S6-19 Section 4.4.4, a seismic performance category is assigned for each bridge based on:

- the site-specific spectral acceleration for a 2% in 50-year probability of exceedance;
- the fundamental period of the bridge, T, in the direction under consideration; and
- the importance category.

Due to the low spectral acceleration values with consideration of the assigned Site Seismic Class D (i.e. F(0.2) x Sa(0.2) and F(1.0) x Sa(1.0)) at the proposed bridge site, Seismic Performance Category (SPC) 1 could be assigned for this bridge regardless of the bridge return period and importance. As noted below Table 4.10 of the CHBDC S6-19, for lifeline bridges in SPC1, detailing of structural elements shall adopt requirements for SPC2 as a minimum.

As per the CHBDC S6-19 Section 4.4.5.1., seismic analysis of bridges in SPC1 is not required. However, design forces for retaining elements and bridge support lengths should meet the requirements specified in the CHBDC S6-19 Sections 4.4.10.2 and 4.4.10.5.

### 8.4.3 Peak Ground Acceleration (PGA)

Seismic hazard values for the Pond Mills Road overpass site were obtained from Natural Resources Canada (2015 National Building Code Canada). Table 8.2 below summarizes the parameters obtained and recommended for use in the design based on a 2475-year return period.

**Table 8.3: Peak Ground Acceleration Data**

| $PGA$ **Site Class C** | $S_a(0.2)$ | $S_a(1.0)$ | $PGA_{\text{ref}}$ | **Site Class** | Site Adjusted $PGA$ |
|---|---|---|---|---|---|
| 0.067g | 0.111g | 0.041g | 0.054g | D | 0.086g |

The 2015 NBC Seismic Hazard calculation sheet is provided in Appendix F.

### 8.4.4 Liquefaction Potential

The potential liquefaction of the site soil under seismic loading conditions was assessed. The evaluation indicated that liquefaction of the foundation soils is not a concern for this site due to:

(a) relatively low seismic hazards, and
(b) relatively high fine content (silty or clayey nature) of the site soils

The presence of less than 3 m thick very loose to loose granular soil layers won't have significant impacts on overall ground behavior under the given relatively low seismic conditions.

# 8.5 REPLACEMENT BRIDGE FOUNDATION ENGINEERING DESIGN INPUT

The design recommendations presented in the following sections have been developed in accordance with the requirements and methods described in the Canadian Highway Bridge Design Code (CHBDC, 2019).

## 8.5.1 Foundation Options

Table 8.3 presents the advantages, disadvantages, relative assessment of cost and the risks/consequences for various foundation options for the pier and abutment foundations of the proposed bridge replacement from a foundations design and constructability perspective:

**Table 8.4: Comparison of Foundation Options for Pond Mills Road Overpass**

| Option | Advantages | Disadvantages | Relative Cost | Risk/Consequences |
|---|---|---|---|---|
| **Driven Steel H Piles** | • Higher geotechnical resistances than spread footings<br>• Ease of construction<br>• Feasible for integral abutments | • Higher construction cost than spread footings<br>• Possible traffic impact due to large crane and pile driving equipment | Medium | • Cobbles and boulders may be encountered in glacially derived soils that could impede pile penetration to required depths<br>• Although pile relaxation is a risk, the delay required for PDA testing will render this potential issue as non-consequential. |
| **Driven Steel Pipe Piles** | • Higher geotechnical resistances than spread footings and driven steel H piles | • Higher construction cost than spread footings<br>• More vibration than driven steel H-piles and not good for the proposed staged construction<br>• More driving problems than Steel H-piles<br>• Possible traffic impact due to large crane and pile driving equipment | Medium | • Cobbles and boulders may be encountered in glacially derived soils that could impede pile penetration to required depths<br>• Although pile relaxation is a risk, the delay required for PDA testing will render this potential issue as non-consequential. |
| **Drilled Caissons** | • Can support/resist higher axial and lateral loads than steel driven piles | • Not suitable for integral abutments<br>• Higher construction cost than other foundation options<br>• Possible traffic impact due to large caisson drilling equipment | High | • Liners and drilling mud likely required due to presence of groundwater.<br>• Use of “wet” installation methods precludes ability to review/confirm materials at the base of the caissons and |

| Option | Advantages | Disadvantages | Relative Cost | Risk/Consequences |
|---|---|---|---|---|
| | | | | assess the potential for reduced capacity |
| **Spreading Footings** | • Ease of construction<br>• Lower foundation costs than deep foundations | • Not suitable for integral abutments<br>• Relatively lower geotechnical capacity than deep foundation<br>• Larger foundation areas required compared to pile caps or drilled caissons<br>• May increase requirements for roadway protection | Low to medium | • Potential excessive settlement under large loads<br>• Increased potential for differential settlement |

Based on the above, the preferred option from a geotechnical/foundations perspective is to support the integral abutments for the proposed bridge structures on driven steel H-piles.

## 8.5.2 Driven Pile Foundations

### 8.5.2.1 Design Considerations

Driven pile foundations consisting of steel H-piles, deriving their load-carrying capacity from both shaft friction and tip resistance, can be used to support the abutments of the proposed replacement bridge structure. Closed-end pipe piles are not recommended as they would displace more soil than H-piles during installation which could lead to deformation/heave of adjacent piles and the adjacent ground during pile installation. Closed-end pipe pile will also generate significant higher vibration than steel H-piles and it is not suggested for the planned staged bridge construction.

The driving of steel H-piles for the new overpass is not expected to adversely affect the existing and newly built structure(s) and approach embankment. However, vibration monitoring should be carried out during the pile driving to confirm this.

### 8.5.2.2 Geotechnical Axial Resistance

**Axial Resistance in Compression**

The axial resistances at Ultimate Limit State (ULS) for driven steel 310x110 were assessed using the Federal Highway Administration (FHWA) and API (American petroleum institute) design methods with the program APILE (Ensoft, 2019). The geotechnical model outlined in Tables 8.1 and 8.2 and on Drawing s No. E1 and No. E2 were used as input to these analyses.

The factored geotechnical resistances at Ultimate Limit States (ULS) outlined in Table 8.4 may be used in design.

### Table 8.5: Factored Geotechnical Resistances at ULS and at SLS – Pile Foundations

| Pile Type | Anticipated Pile Length below pile cap(m) | Pile Tip Elevation[1] (m) | Factored Geotechnical Resistance at ULS[2 & 3] (kN) | Factored Geotechnical Resistance at SLS[2 & 3] (kN) |
|---|---|---|---|---|
| **West Abutment** | | | | |
| HP 310 X 110 | 21.6 | 250 | 1100 | 850 |
| **East Abutment (the south half)** | | | | |
| HP 310 X 110 | 21.3 | 250 | 1100 | 850 |
| **East Abutment (the north half)** | | | | |
| HP 310 X 110 | 21.3 | 250 | 930 | 740 |
| HP 310 X 110 | 26.3 | 245 | 1100 | 850 |

Notes:

[1] Pile lengths and tip elevations are based on the underside of the abutment foundation as provided above in Section 8.1.2.

[2] In accordance with Table 6.1 in the CHBDC, the ULS Geotechnical Resistances were determined based on a consequence level of “Typical” with a consequence factor equal to 1.

[3] In accordance with Table 6.2 in the CHBDC and the site and prediction model understanding classification of “Typical”, a resistance factor of 0.4 (static analysis, compression) has been used in calculating the factored geotechnical resistance at Ultimate Limit State (ULS) and a resistance factor of 0.8 (static analysis, settlement, and lateral deflection) has been used in calculating the factored geotechnical resistance at Serviceability Limit State ($SLS_f$).

Figures E3 and E4 of Appendix D provides a profile of geotechnical axial resistance at ULS in compression for HP310x110. A resistance factor of 0.4 should be applied to the calculated ultimate capacity. The estimated geotechnical reaction at SLS for a 25 mm vertical settlement exceeds the geotechnical reaction at $ULS_f$ given above. This SLS reaction was assumed to be 80% of the $ULS_f$.

It should be noted that 100 blows material were not encountered within the bearing layer (lower silt deposit) at PM-03 at the north half of the east abutment, as such piles driven at this location may need to be longer to achieve the above-mentioned ultimate geotechnical resistance of 2,750 kN (or factored resistance of 1,100 kN) within the very stiff to hard silty clay to clay till. After driving beyond the silt deposit, initially, capacity reduction is expected for these piles, however, the ultimate geotechnical capacity could be achieved at a deeper tip elevation of approximately 245 m (or anticipated pile length of 26.25 m below pile cap).

### 8.5.2.3 Downdrag

The proposed overpass structure will be constructed along the similar alignment as the existing bridge. No significant grade raise is expected within the proposed bridge foundation footprint and existing embankment will be removed to accommodate a longer bridge. In addition, the site soils consist predominantly of dense to very dense granular soils and very stiff to hard cohesive soils and piles are not designed to purely rely on end bearing. Based on these conditions, the piles are not anticipated to be subjected to significant downdrag loads.

### 8.5.2.4 Soil Setup, Relaxation and Pile Capacity Validation

Pile/Soil setup effect is a natural phenomenon where pile load capacity increases over time as the results of dissipation of pore-water pressure. The magnitude of pile/soil setup is governed by three main factors: pile slenderness ratio, elapsed time, and type of surrounding soil.

Piles will be driven through significant thickness of clay/clayey soils at the site. Piles driven in cohesive soils generally gain capacity after driving has been completed and excess pore pressures have dissipated (i.e., the capacity of friction piles in clayey soils increases with time). The $ULS_f$ capacities identified in the previous sections represent the 'long-term' capacities of the piles. Capacities determined by static pile testing or restriking of piles (particularly piles that derive most of their capacity from skin friction) at the time of, or shortly following, driving would not be expected to equal the long-term capacities. To determine the actual, long-term pile capacities the following procedures are recommended to be carried out.

- At each abutment, two of the production piles should be driven to the targeted tip elevation while full-time Pile Driving Analysis (PDA) testing is carried out to obtain the initial drive resistances.
- These 'test piles' shall remain in place for two weeks to allow for 14 days of soil set-up to occur.
- PDA testing of the piles shall be carried out on day 14.
- The result of the day-zero and day-14 results will be used to project the capacities after one year using the following relationship.

$$Q_t = Q_o \left(A \log\left(\frac{t}{t_o}\right) + 1\right)$$ Skov and Denver, 1988

The 'A' constant will be determined based on the setup determined at day-zero and day-14, followed by calculation of $Q_{365}$ which will be considered the long-term capacity of the piles.

For H-piles deriving their capacity predominantly from friction within the very stiff to hard silty clay / clayey silt till, relaxation and reduction of pile capacity is not generally a concern. The delay required for PDA testing should render this concern as non-consequential.

The Hiley Formula as defined on Structural Drawing SS103-11 should be applied to each driven pile to provide a relative comparison between piles where PDA testing is carried out and the remaining piles. The "Hiley Formula Pile Resistance" for all piles shall be submitted to the geotechnical engineer for comparison with the PDA tested piles.

As per the RFP section 2.4.9.5 Foundation Design and Construction and related subsequent bid enquiries (#166 and 176):

*For pile foundation specified to be driven to a specified ultimate resistance, the specified ultimate resistance shall be validated using dynamic formula analysis and high-strain dynamic testing at end of drive (EOD) and retap/restrike after sufficient time has passed to allow soil setup. In each pile group, 10% of the piles rounded up to the next whole number, but no fewer than two piles, shall be re-tapped to confirm that the ultimate axial geotechnical resistance has been achieved and/or sustained. Pile driving records and testing results shall be provided to MTO Foundation Section for information purposes.*

Piles should be supplied and installed/constructed in accordance with the requirements of OPSS.PROV 903 – Construction Specification for Deep Foundations.

The following “Pile Driving Note” should be included on the structural drawings:

- Piles to be driven in accordance with Standard SS 103-11. Projected ultimate capacity based on the 14-day PDA testing must demonstrate an ultimate geotechnical resistance of 2,200 kN per pile (HP 310X110) based on a geotechnical resistance factor of 0.5, but must be driven to EL. 250 m.
- If PDA testing indicates that the capacity will not achieved at EL. 250.0 m, it should be anticipated that piles will need to be further driven to EL. 245.0 m; Should the initial testing reveal this to be the case, the above pile driving note will need to be adjusted at that time.
- In the case of the northern half of the east abutment, the static analysis suggests that a lower resistance maybe encountered at EL. 250 m; 14-day PDA test results should be used to determine if driving to EL. 245.0 m will be required.

The specified resistance load per pile in the note above is dependent on the pile size selected and the structural load planned to be supported on each pile and is equal to two times the factored geotechnical resistance at ULS for the selected pile type.

#### 8.5.2.5 Drivability

The pile driving equipment shall be appropriate to the driving conditions and capable of achieving the design pile capacity. The pile termination or set criteria should be dependent on the pile driving hammer type, helmet, select pile size and length. The set criteria should be established at the time of pile driving once the equipment is decided.

The site soil generally consists of loose to very dense granular soils and stiff to hard cohesive soils including glacial tills (upper clayey silt to silty clay). No early termination/refusal of boreholes than the designated hole depths were noted at the site due to possible cobbles and boulders although some auger grindings, gravel and rock fragments within auger cutting and split spoon samples were noticed during Stantec investigation. Based on the current Stantec investigations’ findings, no significant pile driving issues are anticipated within that soil deposit at the site.

#### 8.5.2.6 Pile Lateral Resistance

##### SLS Resistance Modelling and P-Y Curves

The response of a pile to lateral loads is a non-linear relationship. Non-linear elastic-plastic springs (i.e., p-y curves representing the load intensity per unit length of pile (p) versus the lateral deflection of the pile) can be used in evaluating the structural response of the pile in response to lateral loads.

The program LPile 2019 developed by Ensoft, Inc. (Ensoft, 2019) was used to develop p-y curves for HP 310x110 piles, the preferred pile size for this site. A moment of inertia of 237x10$^6$ mm$^4$ was used for an HP310x110 pile section. A modulus of elasticity of 200 GPa was used for the pile material (steel). The pile was modelled with a total length of 23 m. The unfactored geotechnical input parameters that were used in the analyses for the piles for the west abutment and the east abutment are displayed in the following tables. For an integral abutment, sand fill within the flex zone (CSP) should be considered loose for a lateral resistance assessment.

**Table 8.6: Recommended Parameters for Lateral Pile Capacity Evaluation**

| Soil Layer | Elevation Interval (m) | | Effective Unit weight, $\gamma$ | Friction angle, $\phi$ | Undrained shear strength, $S_u$ |
|---|---|---|---|---|---|
| | From | To | (kN/m$^3$) | (°) | (kPa) |
| Stiff to hard CLAYEY SILT to SILTY CLAY | 268 | 267 | 21.0 | 32 | 200 |
| Stiff to hard CLAYEY SILT to SILTY CLAY | 267 | 262 | 11.2 | 32 | 200 |
| Loose to dense SILT to SANDY SILT | 263 | 256 | 11.2 | 30 | - |
| Stiff to hard SILTY CLAY | 256 | 252 | 10.7 | 32 | 150 |
| Dense to very dense SILT | 252 | 247 | 12.7 | 33 | - |

Note: Groundwater level is assumed at 267.0 m.

The p-y curve values versus depth for the piles size identified above at each foundation unit are presented in Figure E5 and Table E1 in Appendix E. These tables provide a series of curves obtained from the LPILE program generated for selected depths below the pile head. The p-y curves can be used in the structural evaluation of the H-piles noting that the p-y curves provided are unfactored and that appropriate resistance factors (i.e., as outlined in Table 6.2 of the CHBDC, 2019) should be applied when assessing the geotechnical lateral resistances of the piles at ULS and SLS.

##### Group Action

The horizontal resistance of piles should consider the group action of piles (pile interaction) in accordance with Section 6.11.3.4 and the associated commentary of the CHBDC.

Group action of piles (pile interaction) for lateral loading should be considered if centreline spacing of piles is less than 8 pile diameters (or least lateral dimension of pile) parallel to the direction of lateral load or less than 4 pile diameters perpendicular to the load.

The effect of interaction between piles can be considered by applying a reduction factor to the soil resistance (i.e., the p-multiplier) of a single pile to obtain p-y curves for the pile group. The reduction factors to be applied are dependent on the pile spacing/group geometry. The reduction factors (i.e., p-multipliers) outlined in Figures C6.11.3(r), C6.11.3(s) and C6.11.3(t) of Section C6.11.3.4 of the CHBDC should be used. The following reduction factors may be used to account for pile group action:

### Table 8.7: Recommended Reduction Factors for Pile Groups

| Pile spacing / pile diameter | Reduction Factor | Pile spacing / pile diameter | Reduction Factor |
|---|---|---|---|
| **Load Parallel to Pile Spacing** | | **Load Perpendicular to Pile Spacing** | |
| 7 | 1.0 | 4 | 1.0 |
| 4 | 0.8 | 3 | 0.9 |
| 3 | 0.7 | 2 | 0.75 |
| 2 | 0.6 | - | - |

## ULS and SLS Lateral Resistances

At SLS, the horizontal resistance of the pile will be controlled by deflections and the horizontal resistance of the piles should be calculated based on the p-y curves of the soil as discussed above. In general, 10 mm pile deflection under the pile cap is considered for a SLS condition. Both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS.

Based on the p-y analysis, a geotechnical resistance at SLS (10 mm pile top lateral deflection with a resistance factor of 0.8 as per CHBDC S6-19) was assessed for the HP 310 X110 driven to the pile tip elevation mentioned above. A factored ULS lateral resistance for the same pile was also assessed using LPILE. When carrying out p-y based analysis, the ultimate lateral resistance of the pile (ULS) is generally taken as the structural capacity of the pile laterally supported by the p-y springs or a maximum displacement defined by the structural engineer. Where no limiting deformation is applied to the pile head, the LPILE result represents the structural capacity of the pile.

Based on the LPILE analysis carried out using the soil properties provided in Table 8.5, the following unfactored lateral pile capacities have been calculated for a HP310x110 pile with a fixed head condition (as per the MTO Report S0-96-01 Integral Abutment Bridges). No pile axial loads were considered for this analysis.

- Strong axis – 190 kN with a corresponding 10 mm of pile head deformation
- Strong axis – 370 kN with a limiting 50 mm deformation at the pile head
- Weak axis – 110 kN with a corresponding 10 mm of pile head deformation
- Weak axis – 210 kN with a limiting 50 mm deformation at the pile head

A geotechnical resistance factor of 0.5 and 0.8 (as per CHBDC S6-19) should be applied to obtain the lateral resistances at ULS and SLS, respectively. Group reduction factors should also be applied to account for pile group action as necessary.

#### 8.5.2.7 Axial Resistance in Tension

The axial resistance in tension at Ultimate Limit State (ULS) for driven steel 310x110 was assessed using the API (American petroleum institute) design method with the program APILE (Ensoft, 2019). The geotechnical model outlined in tables 8.1 and 8.2 and on Drawing No. D1 and D2 were used as input to the analysis.

For design against uplift, the tensile resistance provided in the following table is recommended.

**Table 8.8: Recommended Uplift Resistance – Pile Foundations**

| Pile Type | Assumed Pile Length (m) | Factored Geotechnical Resistance (Tension) at ULS (kN) |
|---|---|---|
| HP 310 X 110 | 21.3 - 21.6 | 600 |

A resistance factor, $\phi_{gu}$, of 0.3 has been applied to calculate the ULS resistance. The factored geotechnical resistance (tension) at ULS provided above does not include the self-weight of the pile.

#### 8.5.2.8 Other Pile Details

To facilitate pile installation, embankment fill through which piles will be driven must not contain any material with particle sizes greater than 75 mm. Pre-augering may be required through the existing embankment fill if large obstructions are noted during initial construction phase.

Due to the mode of deposition, site soils may contain cobbles and boulders. In order to be able to penetrate boulders, cobbles and hard/very dense zones to achieve the required pile resistance, it is recommended that the pile tips be reinforced as per OPSD 3000.100. Further consideration can also be given to use heavier pile section to minimize potential pile damages.

Piles supporting integral abutments require a minimum 3 m long flex zone which is a CSP filled with loose uniform sand to maintain the pile flexibility. The flex zone sand fill gradation should meet the requirements in the MTO integral abutment Bridges Report SO-96-01.

## 8.6 LATERAL EARTH PRESSURES

### 8.6.1 Abutment Backfill

Ontario Provincial Standard Drawing (OPSD) 3101.150 outlines the required extent of the granular backfill zone at the bridge abutments. The materials used as backfill behind the proposed bridge abutments should consist of free-draining granular fill placed and compacted using methods and equipment appropriate to the type of structure. For the purpose of this report, it is assumed that backfill materials meeting the requirements of OPSS Granular B (Type I or Type II) or Granular A materials will be used.

Excavation and backfill for the new bridge structure should be carried out in accordance with OPSS 902 Construction Specification for Excavation and Backfilling – Structures. Backfill materials should meet the requirements of OPSS.PROV 1010 and be placed and compacted in accordance with the requirements of OPSS.PROV 206 and OPSS.PROV 501, respectively.

### 8.6.2 Static Lateral Earth Pressures

Static lateral earth pressures will need to be considered in the design of abutments, retaining walls (wingwalls) and retained soil systems. These structures should be backfilled using imported free-draining granular fill materials meeting the gradation requirements of OPSS Granular A or Granular B Type I materials.

Computation of earth pressures should be in accordance with Section 6.12 of the CHBDC. For retaining walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressure should be used for design. The effects of compaction should be accounted for by applying a compaction surcharge as outlined in Section 6.12.3 and as shown in Figure 6.8 of the CHBDC. Where applicable (i.e., where unbalanced water pressures may develop), the structures should also be designed to account for hydrostatic pressures.

The total at rest, ($P_O$) active ($P_A$) and passive ($P_P$) thrusts can be calculated using the following equations:

$$P_A = \frac{1}{2} K_a \gamma H^2$$
$$P_O = \frac{1}{2} K_o \gamma H^2$$
$$P_P = \frac{1}{2} K_p \gamma H^2$$

where H is the height of the wall and $\gamma$ is the unit weight of the backfill soil. Values for $K_a$, $K_p$, $K_o$ and $\gamma$ are provided in Table 8.88 for horizontal backfill conditions. These values should be adjusted if sloped backfill is considered. The thrust acts at a point one third up the height of the wall.

**Table 8.9: Recommended Non-Seismic Earth Pressure Parameters (Horizontal Backfill)**

| Parameter | Existing Fill Materials | OPSS Granular B Type I | OPSS Granular A and Granular B Type II |
|---|---|---|---|
| Bulk Unit Weight, $\gamma$ (kN/m$^3$) | 21 | 22 | 22 |
| Effective Friction Angle | 28 | 32° | 35° |
| Coefficient of Earth Pressure at Rest (Ko) | 0.53 | 0.47 | 0.43 |
| Coefficient of Active Earth Pressure (Ka) | 0.36 | 0.31 | 0.27 |
| Coefficient of Passive Earth Pressure (Kp) | 2.77 | 3.25 | 3.70 |

*this granular material should be tested to confirm the friction angle and compacted density as per relevant OPSSs

### 8.6.3 Seismic Lateral Earth Pressures

The following design parameters are provided for use in assessing the earth pressures induced on the bridge abutment and wingwalls under seismic loading conditions.

The total active and passive thrusts under seismic loading conditions can be calculated using the following equations:

$$P_{AE} = ½ K_{AE} \gamma H^2 (1 - k_V)$$

$$P_{PE} = ½ K_{PE} \gamma H^2 (1 - k_V)$$

where:

$K_{AE}$ = active earth pressure coefficient (combined static and seismic)

$K_{PE}$ = passive earth pressure coefficient (combined static and seismic)

$H$ = height of wall

$k_h$ = horizontal acceleration coefficient

$k_v$ = vertical acceleration coefficient

$\gamma$ = total unit weight

For this site, the following design parameters were used to develop the recommended $K_{AE}$ and $K_{PE}$ values as per CHBDC 2019.

**Table 8.10: Seismic Design Parameters to Estimate Lateral Earth Pressures**

| Site Adjusted $\boldsymbol{PGA}$ | Horizontal Acceleration Coefficient, $k_{ho}$ | Horizontal Acceleration Coefficient, $k_h$ |
|---|---|---|
| | Non-Yielding | Yielding $(wall\ movements\ of\ 25\ mm\ to\ 50\ mm)$ |
| 0.0864g | 0.086 | 0.043 |
| Note: $k_{ho}$ is the seismic horizontal acceleration coefficient that corresponds to zero wall movement and is equal to the site-adjusted $\boldsymbol{PGA}$ estimated at ground surface. The vertical acceleration coefficient ($k_v$) should be ignored in the calculations as per CHBDC 2019, section C4.14.7.2. | | |

The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate.

The seismic earth pressures may be calculated using the parameters detailed in Table 8.10 for horizontal backfill configuration. These values should be adjusted if sloped backfill is considered.

**Table 8.11: Recommended Seismic Earth Pressure Parameters (Horizontal Backfill)**

| Parameter | OPSS Gran B Type I | OPSS Gran A and Gran B Type II | Existing Fill Materials |
|---|---|---|---|
| Bulk Unit Weight, $\gamma$ (kN/m3) | 22 | 22 | 21 |
| Effective Friction Angle | 32 | 35 | 28 |
| Passive Earth Pressure, (KPE) | 3.18 | 3.61 | 2.70 |
| Height of Application of PPE from base as a ratio of wall height, (H) | 0.327 | 0.327 | 0.326 |
| **Yielding Wall** | | | |
| Active Earth Pressure ($K_{AE}$) for Yielding Wall | 0.33 | 0.29 | 0.39 |
| Height of Application of $P_{AE}$ from base as a ratio of wall height, (H) for Yielding Wall | 0.353 | 0.354 | 0.352 |
| **Non-Yielding Wall** | | | |
| Active Earth Pressure ($K_{AE}$) for Non-Yielding Wall | 0.36 | 0.32 | 0.42 |
| Height of Application of $P_{AE}$ from base as a ratio of wall height, (H) for Non-Yielding Wall | 0.372 | 0.374 | 0.369 |

## 8.7 APPROACH EMBANKMENT GRADE RAISE AND WIDENING

The maximum height of the existing bridge approach embankment is about 6 m above the surrounding grade and existing embankment side slope is found about 2H:1V. As mentioned earlier, no visible signs of embankment instability and settlement were noted during the site reconnaissance and borehole investigation.

As part of the project, the existing overpass approach embankment will be widened and the grade will be raised. As per the cross-sections provided, 1 m to 2 m grade raise with 3 m to 5 m wide embankment widening (at the embankment crest level, on both sides of the existing embankment) are proposed at each abutment location. The following comments are made on the proposed approach embankment grade raise and widening:

- The proposed embankment widening will be done with typical 2H:1V side slope.
- If overall embankment height will be in excess of 8 m, a mid-slope bench should be provided for maintenance as per OPSD 202.010.
- It is assumed that all embankment widening higher than 4.5 m will be done using OPSS 1010 SSM (or other compactible inorganic granular materials which can have an internal friction angle greater than 30 degrees after placement) and embankment widening will be carried out in accordance with relevant MTO standards such as OPSS.PROV 206 (subgrade preparation embankment construction) and OPSS.PROV 501 (compaction, quality control).

- In area where new fill is abutting to the existing embankment fill, the existing fill surface should be properly benched in accordance with OPSD 208.01.
- To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS.MUNI 802 (Topsoil) and OPSS.PROV 804 (Seed and Cover) should be carried out as soon as possible after widening of the embankments.
- It is also imperative that the designs include provisions for preventing surface water flow on the embankment side slope face. Consideration can be given to using a mountable curb and gutter arrangement to control and divert surface water away from the top of the slope. Surface water must be properly directed to armoured outfalls/outlets designed to drain into road and highway ditches.

For reference, selected Highway embankment cross sections are included in Appendix E.

### 8.7.1 Embankment Stability

Slope stability analyses were carried out at the critical locations of the highway embankments (i.e., section where the embankment is highest and side slope is steepest, right at the west and east abutments) using the commercially available slope stability analysis software, SLOPE/W (GeoStudio 2020). The input geotechnical design parameters are summarized in Table 8.1. A horizontal seismic load coefficient of 0.043g (equal to half the site Adjusted PGA) was used for the seismic/pseudo-static slope stability evaluation.

A minimum factor of safety (FOS) of 1.3 to 1.4 (corresponding to resistance factor 0.7 and 0.75 as per the MTO Provincial Engineering Memorandum # 2020-01 dated March 23, 2020) is considered acceptable against static, deep-seated embankment instability depends on where majority of slip circle is located. For seismic analyses, a minimum FOS of 1.1 is considered acceptable against pseudo-static, deep-seated embankment instability depends on where majority of slip circle is located.

The results of a slope stability analysis of overpass approach embankment are presented on Figures F1 to F4 in Appendix F. The results of these stability analyses indicate that the proposed embankment grade raise and widening with a 2H:1V side slope are acceptable (FOS>1.5 for static and FOS>1.1 for seismic condition).

### 8.7.2 Embankment Settlements

The proposed embankment grade raise & widening will induce settlement of existing embankment fills and native soils (immediate settlement for granular soils and recompression of cohesive soils). A two-dimensional finite element analysis using Rocscience RS 2 (2D finite element analysis) was carried out for the most critical embankment cross section to check the magnitude of settlements across the embankment crest.

The soil parameters provided in Table 8.1 were used and FEM analysis results are presented in Figure F5 in Appendix F. The maximum settlement along the embankment crest is estimated to less than 40 mm. Based on the prevailing subsurface conditions (predominantly granular soils and over-consolidated clayey soils), it is expected that more than half of the estimated settlement will be occurred during the planned staged construction. It will be still beneficial to place all major embankment widening before the winter shutdown period to minimize any long-term settlement potential.

In addition to above settlement, a self-weight settlement of new fill (for the grade raise and embankment widening) should also be considered. Typically, 0.5% of the new fill height is considered as a self-weight settlement amount for well-compacted approved inorganic granular earth fills and it will take about one to two years to compete after construction. Self-weight settlement of well- compacted OPSS 1010 SSM and Granular A and B materials are generally significantly less than inorganic granular earth fill self-weight settlement.

The results of settlement analysis plus self-weight fill settlement after the construction will be generally below the MTO Embankment Settlement Criteria for Design dated July 2010 (total settlement of 50 mm and differential settlement of 200:1 for freeways & longitudinal transitions) and there are no significant settlement concerns for the proposed highway improvement. As per the RFP, embankment and road pavement settlements should be monitored.

## 8.8 CEMENT TYPE AND CORROSION POTENTIAL

The results of an analytical test on two samples of the embankment fill and native soils are presented in Section 5.3 and Appendix D.

The analytical test results of the embankment fill and native soils samples were compared to Table 7.2 of the U.S. Federal Highway Administration Publication No. FHWA-NHI-14-007 (2015) *Table 7.2 Criteria for Assessing Ground Corrosion Potential* for the attack on buried steel. The sulphate concentrations measured in the embankment fill and native soil samples are less than the threshold for non-aggressive soils (less than 200 ppm). However, the concentration of chlorides for the tested fill soil sample (206 ppm) is indicative of an “aggressive" soil (Chloride concentration of more than 100 ppm).

As per the MTO Structural Manual (2021) section 2.8.5, concrete is considered subject to sulphate attack when

- Water-soluble sulphate ($SO_4$) content of the adjacent soil is equal to or greater than 0.10%; or,
- Sulphate ($SO_4$) in groundwater is equal to or greater than 150 mg/L.

When concrete is identified as subject to sulphate attack, the concrete shall be resistant to sulphate attack as required by Special Provision CONC0006. Based on the test results, concrete will not be subject to sulphate attack for this bridge site (water soluble sulphate in soil samples <0.10% which is equivalent to 1000μg/g).

In addition, the analytical test results were compared to *CSA A23.1 Table 3 Additional requirements for concrete subjected to sulphate attack for potential sulphate attack* on concrete. The sulphate concentrations measured in the tested samples are below the exposure class of S-3 (Moderate).

Therefore, based on the two soil samples tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered.

Based on the results of the samples tested and given that the structure is located across the highway and will be exposed to de-icing salt, consideration should be given by the designer to designing for a “C” type exposure class as defined by CSA A23.1 Table 1.

It should be noted that the final selection of exposure class and corrosion mitigation measures should be a decision of the design engineer who takes into account all design considerations including CSA A23.1 Section 4.1.1. durability requirements.

# 9.0 CONSTRUCTION CONSIDERATIONS

## 9.1 CONSTRUCTION STAGING

The construction of the foundations for the new abutments of the overpass is anticipated to involve staging and lane-reductions on Highway 401 using appropriate traffic control. The use of a temporary roadway protection system may be required based on the staging plan.

## 9.2 TEMPORARY PROTECTION SYSTEMS

Temporary protection systems (TPS) may be required to protect traffic on Highway 401 or maintain traffic on Pond Mills Road during construction of the approach embankments and new overpass foundation infrastructure.

The contractor will ultimately be responsible for developing and implement a roadway protection system meeting the requirements of OPSS.PROV 539, including establishing appropriate geotechnical design parameters. The soil parameters provided in Tables 8.1 and 8.8 could be used for design purpose.

The following table compares the available roadway protection options considered for the proposed rehabilitation:

**Table 9.1: Comparison of Roadway Protection Systems**

| Option | Advantages | Disadvantages | Relative Cost | Risk & Consequences |
|---|---|---|---|---|
| Soldier Piles with timber lagging; struts/rakers or tie-backs/anchors | • Relatively simple installation process | • Additional labour required<br>• Groundwater seepage into the excavation can occur without groundwater control<br>• Removal of soldier piles can be difficult | Low | • Potential for groundwater seepage and loss of ground unless groundwater control measures are implemented |

| Option | Advantages | Disadvantages | Relative Cost | Risk & Consequences |
|---|---|---|---|---|
| | | | | • Potential for minor loss of ground at rear of lagging |
| Steel sheet piles (SSP) with/without tie-backs/anchors | • Relatively Simple installation process<br>• Provides cut-off to groundwater seepage | • Difficult to drive/install in soils where cobbles/boulders are present<br>• May require large sections where cantilever design is adopted<br>• More efforts will be required to remove or cut the temporary shoring system | Medium | • Potential for sheet piles to either be damaged, deflected or meet refusal due to obstructions |

Both of the temporary support systems described in the table are considered feasible for use. The temporary support systems should be supported with struts or rakers from the construction side or tie-backs/ground anchors.

Roadway protection design should generally meet the requirements of Performance Level 2 in accordance with DB SP 539 and should consider traffic loading. Performance Level 2 specifies a Maximum Angular Distortion of 1:200 and a Maximum Horizontal Displacement of 25 mm. Strut, raker, or tieback design, if and as required, must be designed not to exceed these limits. Horizontal movement of the temporary roadway protection system should be monitored throughout the bridge replacement process as described in DB SP 539. If more stringent temporary excavation support performance criteria is considered to be necessary for the proposed staged construction immediately next to the existing and newly built bridge structures, a roadway protection design should be developed in accordance with relevant performance levels of DB SP 539.

## 9.3 EXCAVATION AND BACKFILLING

Excavation and backfilling for the new bridge structure should be carried out in accordance with OPSS.PROV 902 Construction Specification for Excavation and Backfilling – Structures.

Any vegetation, fill, organic soils, and other deleterious materials must be removed from beneath the foundation and retaining wall footprints. Where deleterious materials are encountered within the foundation footprint, the materials should be excavated, removed, and replaced with compacted granular fill material.

All side slopes for open cut excavations should conform to the Occupational Health and Safety Act regulations for Construction Projects (OHSA). The construction of the new abutments will require excavation through the existing highway pavement structure and underlying fill materials and native soils.

The construction of the new overpass abutments will also require excavation through the existing materials in at the Pond Mills Road and additional fill material placed for the proposed embankment widening and grade raise.

The fill in the existing approach embankment lower portion is likely to consist of general/variable earth fill. The fill in the new approach embankments is also anticipated to consist of general earth fill (both granular and cohesive fills). The granular fill materials are expected to have a compact to dense relative density and the cohesive fill materials at the site to have a generally stiff to very stiff consistency. The underlying native soils consist of generally stiff to very stiff clayey silt to silty clay and compact to dense sandy silt to silty sand. Where space permits, these excavations may be developed using open-cut methods. The fill materials (above the water table) and the native soils above groundwater table would be classified as Type 3 soils.

OHSA indicates that temporary excavations made within Type 3 soils that are above the water table and/or dewatered prior to excavation should be developed with side slopes no steeper than 1H:1V.

Grading work should be carried out in accordance with OPSS.PROV 206 Construction Specification for Grading and SP 206S03. For the proposed embankment widening, the new fill materials should be benched into the existing embankments in accordance with OPSD 208.010.

## 9.4 UNWATERING (GROUNDWATER CONTROL)

The groundwater level was measured at elevations of approximately 266.5 m to 267.0 m in the previous investigation at the site. These elevations are about 1.1 m to 1.6 m below the Pond Mills Avenue grade at Highway 401 centreline.

Excavation required for the new abutment foundation and removal of existing structure will likely be above the static groundwater level. Temporary unwatering, using conventional sump and pump techniques, should be anticipated to be required for excavations and should be satisfactory to handle seepage and infiltration into excavations in the underlying native clayey silt to silty clay deposit.

All groundwater control systems required for the construction of the replacement bridge should be designed and implemented in accordance with NSSP FOUN0003.

Ultimately, the design of dewatering/unwatering systems is the responsibility of the contractor. Depending on the water taking/dewatering volumes and source(s) of water, the dewatering activities may require a Permit to Take Water (PTTW) from the Ministry of Environment, Conservation and Parks (MECP) or registration of the water taking activity in the Environmental Activity and Sector Registry (EASR). The permit/registration requirements are outlined in Table 1.0 of CDED B517.

## 9.5 INSTRUMENTATION AND MONITORING

An Instrumentation and Monitoring Plan should be prepared at least 3 months prior to commencement of earthworks for the construction widening of the approach embankments and overpass replacement. The Plan should include the following:

- Monitoring before, during and after construction to check the safety of the work
- Potential impacts of proposed construction on surrounding facilities
- Check compliance with performance specifications
- Assess design assumptions and refine estimates of future performance
- Monitoring before, during and after construction to check the safety of the work
- Discussion of potential for ground movements and impacts to Pond Mills Road, Highway 401, existing and newly built bridge structures;
- Construction vibration monitoring;
- Buried utility (e.g. watermain and gas) monitoring within the earthwork zone of influence;
- Temporary protection system monitoring as per DB SP 539.
- Settlement surveys should be carried out before, during, and following construction. As a minimum, monitoring is expected to include survey points along the existing road surface and on the existing bridge abutments. Post-construction differential settlement between abutments and abutment approaches should be taken at months 3, 6, 12, 18 and 24 of the general warranty period, starting immediately after paving is complete; elevations at the centreline of each lane should be measured at all bridge abutments, and at distances of 20 m, 50 m, 75 m, and 100 m from the abutments.

# 10.0 SPECIFICATIONS

The following specifications are referenced in this report:

**Table 10.1: Specifications Referenced in the Report**

| Document | Title |
|---|---|
| NSSP FOUN0003 | Dewatering Structure Excavations |
| OPSS.PROV 206 | Grading |
| OPSD 202.010 | Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankment |
| OPSD 208.010 | Benching of Earth Slopes |
| OPSS.PROV 212 | Construction Specification for Earth Borrow |
| OPSD 3000.100 | Foundation, Piles, Steel H-Pile Driving Shoe |
| OPSD 3090.101 | Foundation Frost Depths for Southern Ontario |
| OPSD 3101.150 | Walls, abutment, backfill – Minimum Granular Requirements |
| OPSS.PROV 206 | Construction Specification for Grading |
| OPSS.PROV 501 | Construction Specification for Compacting |
| OPSS.PROV 539 | Construction Specification for Temporary Protection System |
| OPSS.PROV 902 | Construction Specification for Excavation and Backfilling – Structures |
| OPSS.MUNI 802 | Construction Specification for Topsoil |
| OPSS.MUNI 804 | Construction Specification for Seed and Cover |
| OPSS.PROV 804 | Construction Specification for Temporary Erosion Control |
| OPSS 805 | Construction Specification for Temporary Erosion and Sediment Control Measures |
| OPSS.PROV 903 | Construction Specification for Deep Foundations |
| OPSS.PROV 1010 | Material Specification for Aggregates |
| SP517F01 | Amendment to OPSS 517, July 2017 |
| SP105S10 | Construction Specification for Compaction |
| SP109S12 | Amendment to OPSS 902, November 2010 |
| SP 206S03 | Earth Excavation, Grading |
| SP 599S22 | Retained Soil System, (Design and Construction Requirements) |
| SP 599S23 | Retained Soil System (Requirements for Materials and QC/QA testing) |
| DB SP 539 | Amendment to OPSS 539 |
| DB SP 902 | Amendment to OPSS 902 |
| DB SP 903 | Amendment to OPSS 903 |
| SP BRDG0007 | CSP for Integral Abutment |

# 11.0 MISCELLANEOUS

The field work was carried out under the supervision of Mr. Wuhib Tamrat, Mr. Akshat Shukla, Mr. Justin Moleta, and Mr. Binoy Debnath, under the direction of Mr. Gwangha Roh, Ph.D., P. Eng.

The drilling equipment was supplied and operated by Landshark Drilling based in Brantford and DBW Drilling Inc. based in North York, Ontario. Traffic control service was provided by CRH Group Inc. Chemical testing for pH, soluble sulphate, and chloride content, and resistivity was out by Agat Laboratories based in Mississauga.

The location and elevation survey of the investigation holes was carried out by Stantec’s Geomatics Group based in London. Geotechnical laboratory testing was carried out at Stantec’s Markham laboratory.

This report was prepared Ramin Ghassemi, Ph.D., P.Eng. and reviewed by Gwangha Roh Ph.D., P.Eng. and by Raymond Haché, M.Sc., P. Eng., Designated Principal MTO Foundation Contact.

# 12.0 CLOSURE

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review our recommendations when the drawings and specifications are complete.

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information and its effects on the above recommendations.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

Respectfully submitted,

**STANTEC CONSULTING LTD.**

LICENSED PROFESSIONAL ENGINEER
R. GHASSEMI
100536678
2023/02/02
PROVINCE OF ONTARIO

Ramin Ghassemi

Ramin Ghassemi, Ph.D., P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
G. H. ROH
100148471
2023/02/02
PROVINCE OF ONTARIO

Gwangha Roh, Ph.D., P. Eng.
Senior Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
J.G.A.R. HACHÉ
17713504
2023/02/02
PROVINCE OF ONTARIO

Raymond Haché, M.Sc., P.Eng.
Senior Principal, Designated Principal MTO Foundation Contact

\\cd1215-f01\work_group\01216\active\1650\165001239\reports\reports\08-pond mills\final fidr\165001239_darft_hwy401highbury_fidr_pondmills_20230131.docx

# 13.0 REFERENCES

American Association of State Highway and Transportation Officials. 2012. AASHTO LFRD Bridge Design Specifications, Washington DC.

ASTM. 1999. Standard Test Methods for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). ASTM International, West Conshohocken, PA.

ASTM. 2000. Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) (ASTM D2487). ASTM International, West Conshohocken, PA.

Canadian Standards Association. 2014. Standard A23-1.14/A23.2-14. Concrete Materials and Methods of Concrete Construction / Test Methods and Standard Practices for Concrete.

CFEM. 2006. Canadian Foundation Engineering Manual, Fourth Edition. Canadian Geotechnical Society, 488 p

Chapman, L.J. and D.F. Putnam. 1984. The Physiography of Southern Ontario, Ontario Geologic Survey

CHBDC. 2019. Canadian Highway Bridge Design Code. Canadian Standards Association, Mississauga, Ontario.

CHBDC. 2006. Canadian Highway Bridge Design Code. Canadian Standards Association, Mississauga, Ontario.

CHBDC. 2014. Canadian Highway Bridge Design Code. Canadian Standards Association, Mississauga, Ontario.

Dreimanis, A., 1963: Pleistocene Geology of the St. Thomas Area (East Half), Southern Ontario. Ontario Department of Mines, Preliminary Geological Map 238, scale 1:50,000.

Ensoft. 2007. A Program for the Analysis of the Axial Capacity of Driven Piles. APILE Plus Version 5.0. Ensoft, Inc., Austin, Texas.

Ensoft. 2010. User's Manual for Computer Program LPILE Plus Version 6.0. Ensoft, Inc., Austin, Texas.

Kanagaratnam Balachandran et al. 2017. Statistical Correlations Between Undrained Shear Strength (Cu) and both SPT-N value and Net Limit Pressure (PL) for Cohesive Glacial Tills, Geo-Ottawa 2017.

NBC. 2015. National Building Code of Canada Vol.1. National Research Council of Canada, Ottawa, Ontario.

OHSA. 2015. Occupational Health and Safety Act Regulations for Construction Projects. Carswell, Toronto Ontario

Ontario Geological Survey. 1982. Paleozoic Geology of the Brockville – Mallorytown Area

Ontario Geological Survey. 2010. Surficial Geology of Southern Ontario GIS data set.

Ontario Ministry of Transportation (MTO). 2010. MTO Embankment Settlement Criteria for Design.

Ontario Ministry of Transportation (MTO). 2014. Structural Manual. Bridge Office, St. Catharines, Ontario.

Ontario Ministry of Transportation (MTO) Report EM-48. 1993. Pile Load and Extraction Tests 1954 - 1992.

Ontario Ministry of Transportation (MTO). 2008. RSS Design Guidelines. Ontario Ministry of Transportation, Engineering Standards Branch.

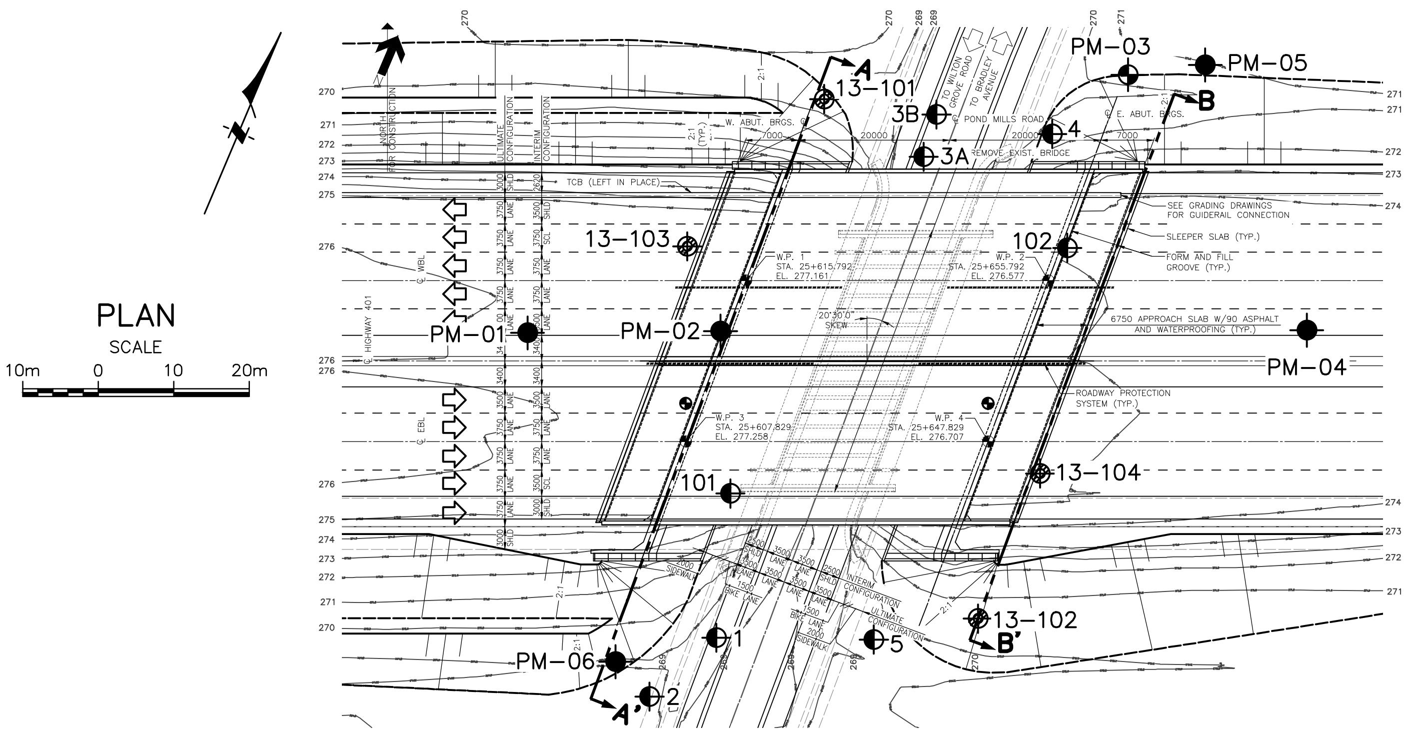
Sanford, B.V., 1969: Geology, Toronto-Windsor Area, Ontario. Geological Survey of Canada, Map 1263A, scale 1:250,000.

Tony Sangiuliano at el. 2018. Axial Resistance Gain of Piles Driven into Artesian Soils: Case Studies, Geo-Edmonton 2018.

# APPENDIX A

## A.1 DRAWING NOS. 1 AND 2 – BOREHOLE LOCATION PLAN AND SOIL STRATA PLOTS

## PLAN



# METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

PLATE No

**CONT 2022–3004**
**WP 3032–11–00**

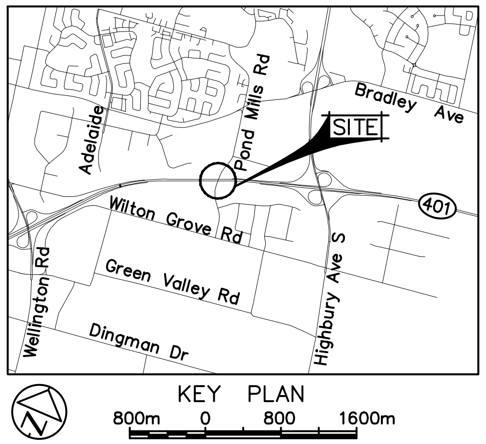
POND MILLS ROAD
OVERPASS
BORHOLE LOCATIONS & SOIL STRATA

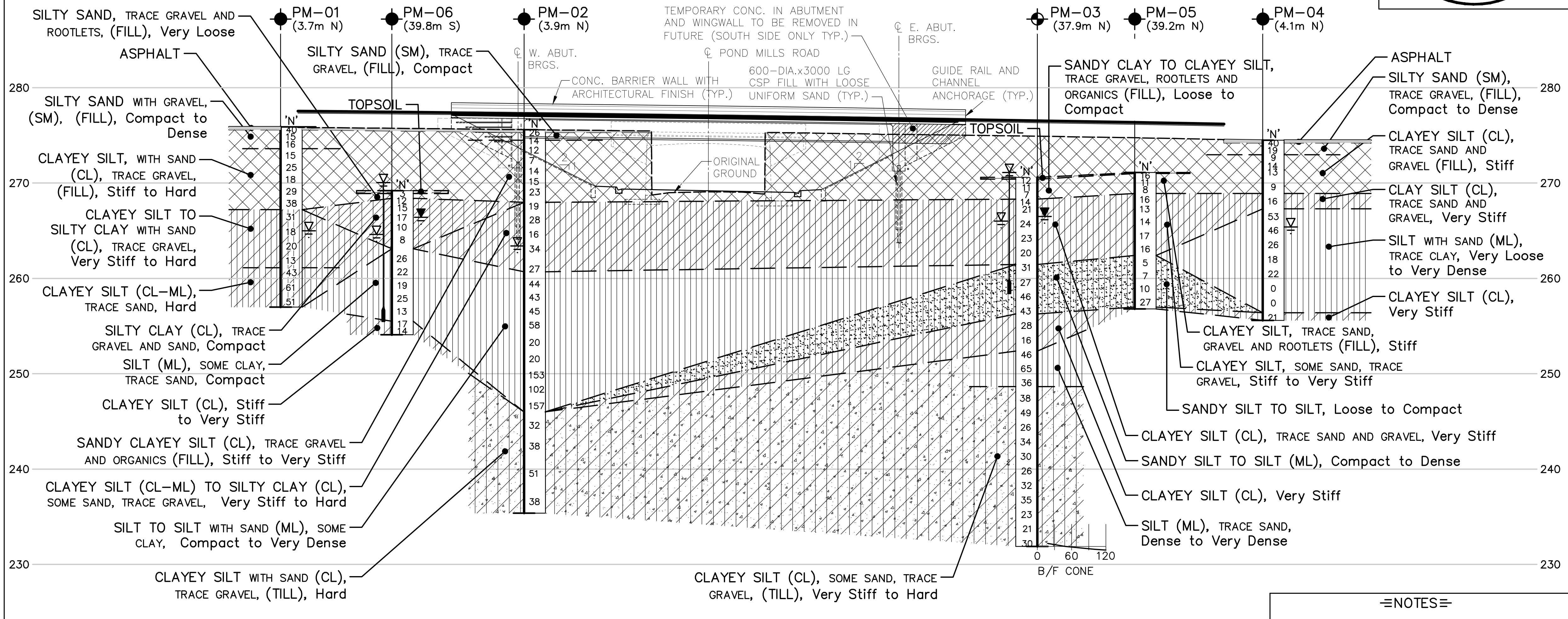
SHEET
–

Stantec

LICENSED PROFESSIONAL ENGINEER
G. H. ROH
100148471
2023-02-01
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
J.G.A.R. HACHÉ
2023-02-01
PROVINCE OF ONTARIO





## PROFILE ALONG HIGHWAY 401 CENTRELINE

SCALE
5m 0 5 10m

## LEGEND

| | |
|---|---|
| ● | Borehole (Stantec, 2022) |
| ⊕ | Borehole & Cone (Stantec, 2022) |
| ⊕ | Borehole (Golder, 2013) |
| ⊕ | Borehole (MTO, 1986) |
| (x.x m) | Offset from Centreline of Alignment |
| N | Blows/0.3m (Std Pen Test, 475 J/blow) |
| ▽ | WL at time of investigation February 2013 and July 2022 |
| ▼ | WL Measured on June 2013 and September 2022 |
| ▮ | Piezometer |

| No | ELEV | MTM ZONE 11 COORDINATES NORTH | EAST |
|---|---|---|---|
| PM–01 | 275.9 | 4 755 685.0 | 411 524.8 |
| PM–02 | 275.6 | 4 755 695.2 | 411 548.2 |
| PM–03 | 270.6 | 4 755 747.4 | 411 584.7 |
| PM–04 | 274.5 | 4 755 725.5 | 411 619.5 |
| PM–05 | 271.1 | 4 755 752.6 | 411 593.6 |
| PM–06 | 269.2 | 4 755 649.5 | 411 552.5 |
| 13–101 | 269.7 | 4 755 728.8 | 411 548.9 |
| 13–102 | 270.7 | 4 755 673.4 | 411 594.4 |
| 13–103 | 275.6 | 4 755 703.8 | 411 539.8 |
| 13–104 | 275.1 | 4 755 694.3 | 411 594.5 |
| 1 | 268.3 | 4 755 657.6 | 411 563.5 |
| 2 | 269.0 | 4 755 647.0 | 411 558.4 |
| 3A | 269.0 | 4 755 726.9 | 411 564.0 |
| 3B | 269.0 | 4 755 732.7 | 411 563.4 |
| 4 | 269.5 | 4 755 736.3 | 411 578.5 |
| 5 | 269.5 | 4 755 665.5 | 411 582.8 |
| 101 | 274.7 | 4 755 675.9 | 411 557.8 |
| 102 | 274.5 | 4 755 723.2 | 411 586.2 |

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102–2 of Form 100.

## ≡NOTES≡

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

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

| REVISIONS DATE | BY | DESCRIPTION |
|---|---|---|
| | | |
| | | |

GEOCRES No 40I14–204

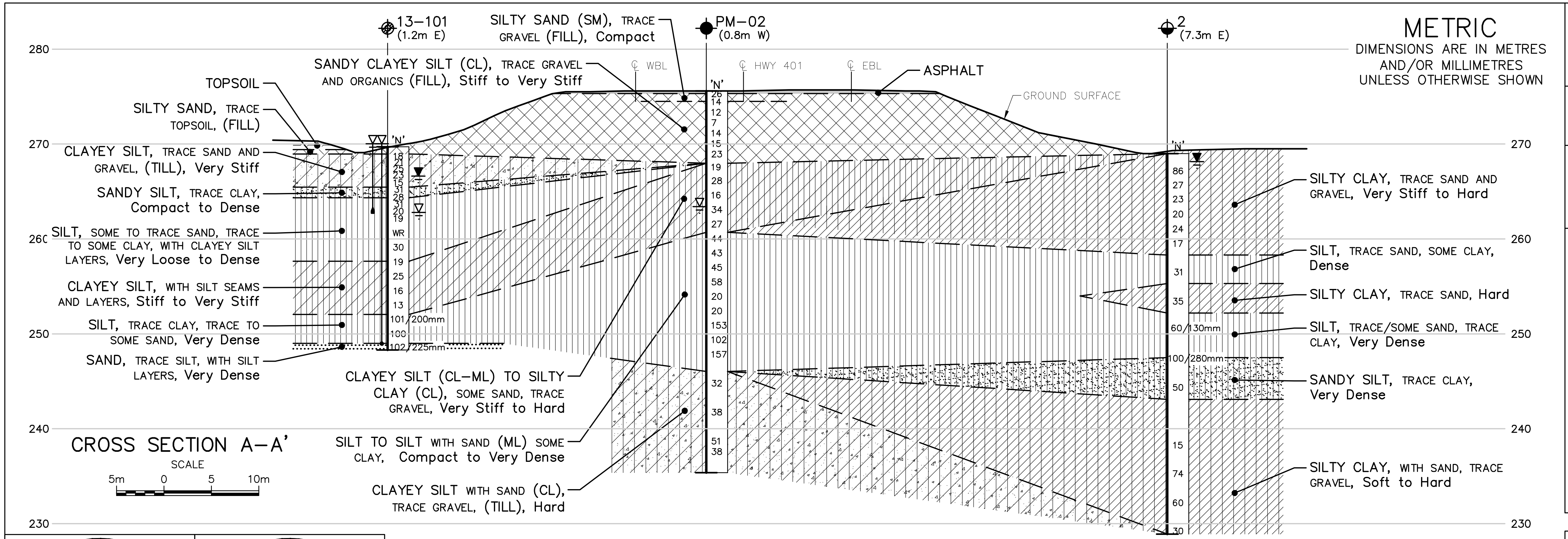
| | | | |
|---|---|---|---|
| HWY No 401 | | | DIST |
| SUBM'D GR | CHECKED | DATE 2023–01–27 | SITE 19X–0372/B0 |
| DRAWN GBB | CHECKED | APPROVED | DWG 1 |

DRAWING NAME: 165001239_Pond Mills Road_P&CS_20230127.dwg
CREATED BY: GBB
MODIFIED:
C:\Users\gbriones\appdata\local\temp\AcPublish_13280\165001239_Pond Mills Road_P&CS_20230127.dwg (Plan) Printed: Jan 27, 2023

MINISTRY OF TRANSPORTATION, ONTARIO PR–D–707 88–05

# METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

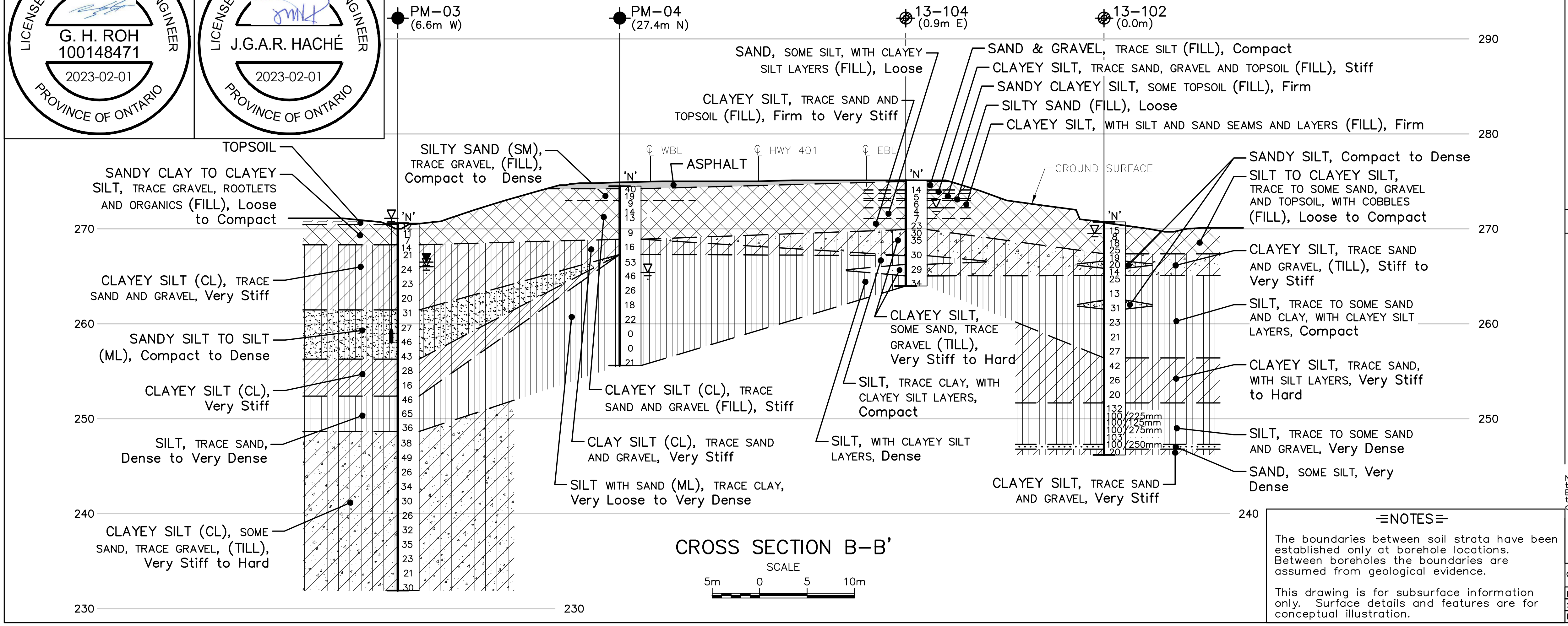


## CROSS SECTION A–A'

SCALE
5m 0 5 10m

G. H. ROH
100148471
2023-02-01
LICENSED PROFESSIONAL ENGINEER
PROVINCE OF ONTARIO

J.G.A.R. HACHÉ
2023-02-01
LICENSED PROFESSIONAL ENGINEER
PROVINCE OF ONTARIO



## CROSS SECTION B–B'

SCALE
5m 0 5 10m

=NOTES=

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

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

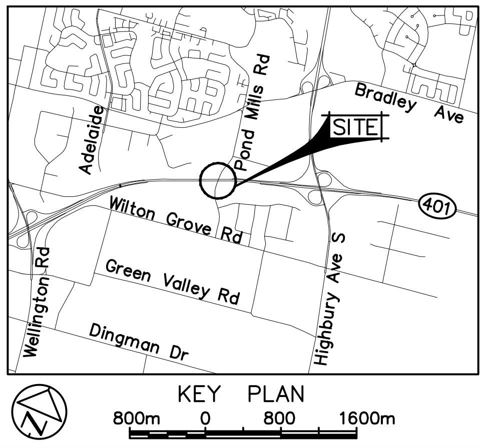
PLATE No

CONT 2022–3004
WP 3032–11–00

POND MILLS ROAD
OVERPASS
SOIL STRATA

SHEET
–

Stantec



KEY PLAN
800m 0 800 1600m

## LEGEND

| Symbol | Description |
|---|---|
| ● | Borehole (Stantec, 2022) |
| ⊕ | Borehole & Cone (Stantec, 2022) |
| ⊕ | Borehole (Golder, 2013) |
| ⊕ | Borehole (MTO, 1986) |
| (x.x m) | Offset from Centreline of Alignment |
| N | Blows/0.3m (Std Pen Test, 475 J/blow) |
| ▽ | WL at time of investigation February 2013 and July 2022 |
| ▼ | WL Measured on June 2013 and September 2022 |
| ▮ | Piezometer |

| No | ELEV | MTM ZONE 11 COORDINATES NORTH | EAST |
|---|---|---|---|
| PM–01 | 275.9 | 4 755 685.0 | 411 524.8 |
| PM–02 | 275.6 | 4 755 695.2 | 411 548.2 |
| PM–03 | 270.6 | 4 755 747.4 | 411 584.7 |
| PM–04 | 274.5 | 4 755 725.5 | 411 619.5 |
| PM–05 | 271.1 | 4 755 752.6 | 411 593.6 |
| PM–06 | 269.2 | 4 755 649.5 | 411 552.5 |
| 13–101 | 269.7 | 4 755 728.8 | 411 548.9 |
| 13–102 | 270.7 | 4 755 673.4 | 411 594.4 |
| 13–103 | 275.6 | 4 755 703.8 | 411 539.8 |
| 13–104 | 275.1 | 4 755 694.3 | 411 594.5 |
| 1 | 268.3 | 4 755 657.6 | 411 563.5 |
| 2 | 269.0 | 4 755 647.0 | 411 558.4 |
| 3A | 269.0 | 4 755 726.9 | 411 564.0 |
| 3B | 269.0 | 4 755 732.7 | 411 563.4 |
| 4 | 269.5 | 4 755 736.3 | 411 578.5 |
| 5 | 269.5 | 4 755 665.5 | 411 582.8 |
| 101 | 274.7 | 4 755 675.9 | 411 557.8 |
| 102 | 274.5 | 4 755 723.2 | 411 586.2 |

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102–2 of Form 100.

| REVISIONS | | |
|---|---|---|
| DATE | BY | DESCRIPTION |

GEOCRES No 40I14–204

| | | | |
|---|---|---|---|
| HWY No 401 | | | DIST |
| SUBM'D GR | CHECKED | DATE 2023–01–27 | SITE 19X–0372/B0 |
| DRAWN GBB | CHECKED | APPROVED | DWG 2 |

PR-D-707 88-05
MINISTRY OF TRANSPORTATION, ONTARIO

Printed: Jan 27, 2023

DRAWING NAME: 165001239_Pond Mills Road_P&CS_20230127.dwg
CREATED BY: GBB
MODIFIED:
C:\Users\gbriones\appdata\local\temp\AcPublish_13280\165001239_Pond Mills Road_P&CS_20230127.dwg (Sections)

# APPENDIX B

## B.1 GEOCRES N0. 40L4-111 (EXPLANATION OF TERMS USED IN REPORT, LABORATORY RESULTS, RECORDS OF BOREHOLE, BOREHOLE LOCATIONS AND SOIL STRATA)

## B.2 GEOCRES N0. 40I14-157 (LIST OF ABBREVIATIONS AND SYMBOLS, RECORDS OF BOREHOLE, BOREHOLE LOCATIONS AND SOIL STRATA, LABORATORY TEST DATA)

# EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

**CONSISTENCY:** COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ($c_u$) AS FOLLOWS:

| $c_u$ (kPa) | 0 - 12 | 12 - 25 | 25 - 50 | 50 - 100 | 100 - 200 | >200 |
|---|---|---|---|---|---|---|
| | *VERY SOFT* | *SOFT* | *FIRM* | *STIFF* | *VERY STIFF* | *HARD* |

**DENSENESS:** COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

| N (BLOWS/0.3m) | 0 - 5 | 5 - 10 | 10 - 30 | 30 - 50 | >50 |
|---|---|---|---|---|---|
| | *VERY LOOSE* | *LOOSE* | *COMPACT* | *DENSE* | *VERY DENSE* |

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

**RECOVERY:** SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

**MODIFIED RECOVERY:** SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

| RQD (%) | 0 - 25 | 25 - 50 | 50 - 75 | 75 - 90 | 90 - 100 |
|---|---|---|---|---|---|
| | *VERY POOR* | *POOR* | *FAIR* | *GOOD* | *EXCELLENT* |

**JOINTING AND BEDDING:**

| SPACING | 50mm | 50 - 300mm | 0.3m - 1m | 1m - 3m | >3m |
|---|---|---|---|---|---|
| JOINTING | *VERY CLOSE* | *CLOSE* | *MOD. CLOSE* | *WIDE* | *VERY WIDE* |
| BEDDING | *VERY THIN* | *THIN* | *MEDIUM* | *THICK* | *VERY THICK* |

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

| | | | |
|---|---|---|---|
| S S | SPLIT SPOON | T P | THINWALL PISTON |
| W S | WASH SAMPLE | O S | OSTERBERG SAMPLE |
| S T | SLOTTED TUBE SAMPLE | R C | ROCK CORE |
| B S | BLOCK SAMPLE | P H | T W ADVANCED HYDRAULICALLY |
| C S | CHUNK SAMPLE | P M | T W ADVANCED MANUALLY |
| T W | THINWALL OPEN | F S | FOIL SAMPLE |

### STRESS AND STRAIN

| | | |
|---|---|---|
| $u_w$ | kPa | PORE WATER PRESSURE |
| $r_u$ | 1 | PORE PRESSURE RATIO |
| $\sigma$ | kPa | TOTAL NORMAL STRESS |
| $\sigma'$ | kPa | EFFECTIVE NORMAL STRESS |
| $\tau$ | kPa | SHEAR STRESS |
| $\sigma_1, \sigma_2, \sigma_3$ | kPa | PRINCIPAL STRESSES |
| $\epsilon$ | % | LINEAR STRAIN |
| $\epsilon_1, \epsilon_2, \epsilon_3$ | % | PRINCIPAL STRAINS |
| E | kPa | MODULUS OF LINEAR DEFORMATION |
| G | kPa | MODULUS OF SHEAR DEFORMATION |
| $\mu$ | 1 | COEFFICIENT OF FRICTION |

### MECHANICAL PROPERTIES OF SOIL

| | | |
|---|---|---|
| $m_v$ | $kPa^{-1}$ | COEFFICIENT OF VOLUME CHANGE |
| $C_c$ | 1 | COMPRESSION INDEX |
| $C_s$ | 1 | SWELLING INDEX |
| $C_\alpha$ | 1 | RATE OF SECONDARY CONSOLIDATION |
| $c_v$ | $m^2/s$ | COEFFICIENT OF CONSOLIDATION |
| H | m | DRAINAGE PATH |
| $T_v$ | 1 | TIME FACTOR |
| U | % | DEGREE OF CONSOLIDATION |
| $\sigma'_{vo}$ | kPa | EFFECTIVE OVERBURDEN PRESSURE |
| $\sigma'_p$ | kPa | PRECONSOLIDATION PRESSURE |
| $\tau_f$ | kPa | SHEAR STRENGTH |
| $c'$ | kPa | EFFECTIVE COHESION INTERCEPT |
| $\phi'$ | -° | EFFECTIVE ANGLE OF INTERNAL FRICTION |
| $c_u$ | kPa | APPARENT COHESION INTERCEPT |
| $\phi_u$ | -° | APPARENT ANGLE OF INTERNAL FRICTION |
| $\tau_R$ | kPa | RESIDUAL SHEAR STRENGTH |
| $\tau_r$ | kPa | REMOULDED SHEAR STRENGTH |
| $S_t$ | 1 | SENSITIVITY $= \frac{c_u}{\tau_r}$ |

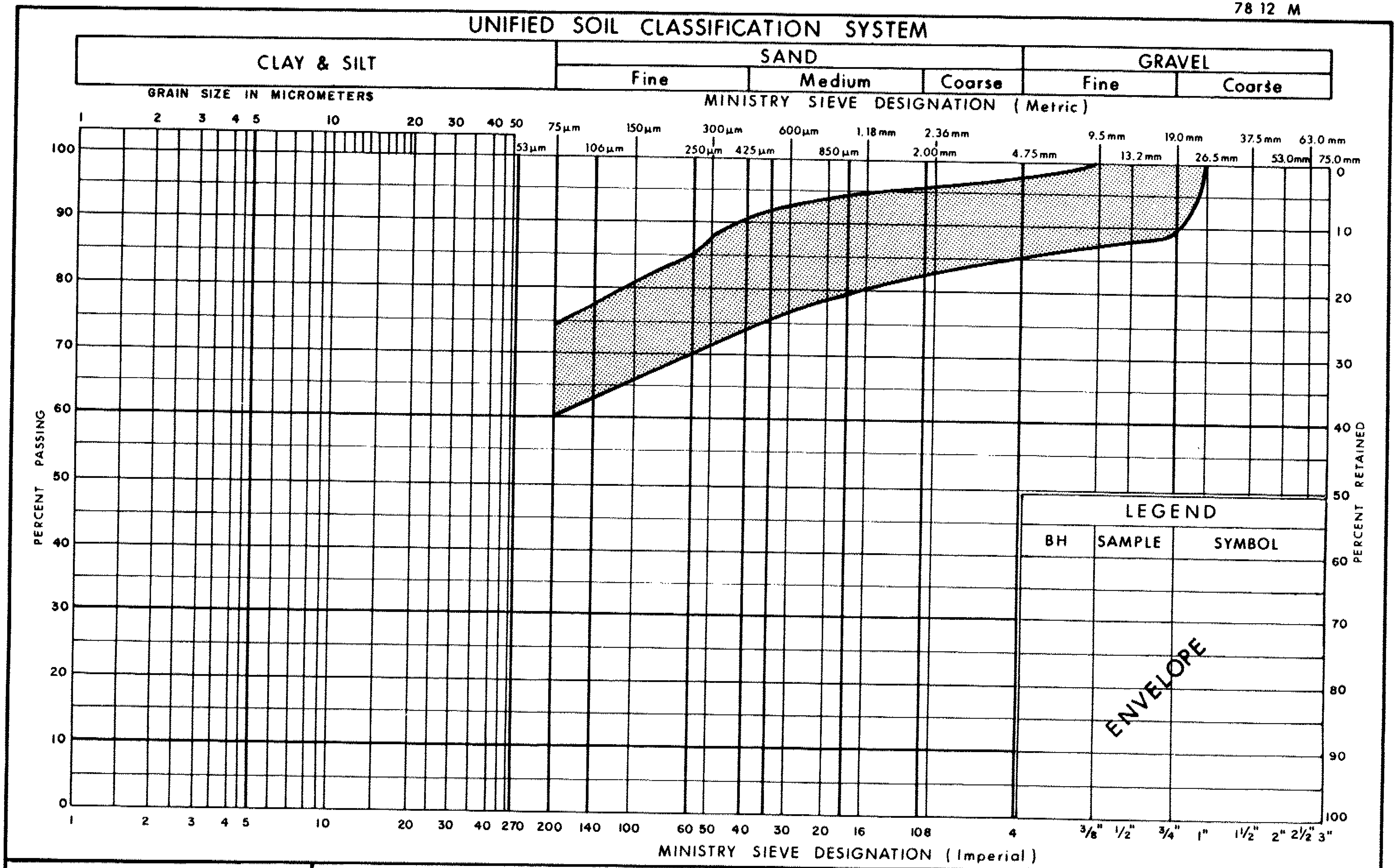
### PHYSICAL PROPERTIES OF SOIL

| | | |
|---|---|---|
| $\rho_s$ | $kg/m^3$ | DENSITY OF SOLID PARTICLES |
| $\gamma_s$ | $kN/m^3$ | UNIT WEIGHT OF SOLID PARTICLES |
| $\rho_w$ | $kg/m^3$ | DENSITY OF WATER |
| $\gamma_w$ | $kN/m^3$ | UNIT WEIGHT OF WATER |
| $\rho$ | $kg/m^3$ | DENSITY OF SOIL |
| $\gamma$ | $kN/m^3$ | UNIT WEIGHT OF SOIL |
| $\rho_d$ | $kg/m^3$ | DENSITY OF DRY SOIL |
| $\gamma_d$ | $kN/m^3$ | UNIT WEIGHT OF DRY SOIL |
| $\rho_{sat}$ | $kg/m^3$ | DENSITY OF SATURATED SOIL |
| $\gamma_{sat}$ | $kN/m^3$ | UNIT WEIGHT OF SATURATED SOIL |
| $\rho'$ | $kg/m^3$ | DENSITY OF SUBMERGED SOIL |
| $\gamma'$ | $kN/m^3$ | UNIT WEIGHT OF SUBMERGED SOIL |

| | | |
|---|---|---|
| e | 1,% | VOID RATIO |
| n | 1,% | POROSITY |
| w | 1,% | WATER CONTENT |
| $S_r$ | % | DEGREE OF SATURATION |
| $w_L$ | % | LIQUID LIMIT |
| $w_p$ | % | PLASTIC LIMIT |
| $w_s$ | % | SHRINKAGE LIMIT |
| $I_p$ | % | PLASTICITY INDEX $= w_L - w_p$ |
| $I_L$ | 1 | LIQUIDITY INDEX $= \frac{w - w_p}{I_p}$ |
| $I_C$ | 1 | CONSISTENCY INDEX $= \frac{w_L - w}{I_p}$ |
| $e_{max}$ | 1,% | VOID RATIO IN LOOSEST STATE |

| | | |
|---|---|---|
| $e_{min}$ | 1,% | VOID RATIO IN DENSEST STATE |
| $I_D$ | 1 | DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$ |
| D | mm | GRAIN DIAMETER |
| $D_n$ | mm | n PERCENT - DIAMETER |
| $C_U$ | 1 | UNIFORMITY COEFFICIENT |
| h | m | HYDRAULIC HEAD OR POTENTIAL |
| q | $m^3/s$ | RATE OF DISCHARGE |
| v | m/s | DISCHARGE VELOCITY |
| i | 1 | HYDRAULIC GRADIENT |
| k | m/s | HYDRAULIC CONDUCTIVITY |
| j | $kN/m^3$ | SEEPAGE FORCE |

78 12 M



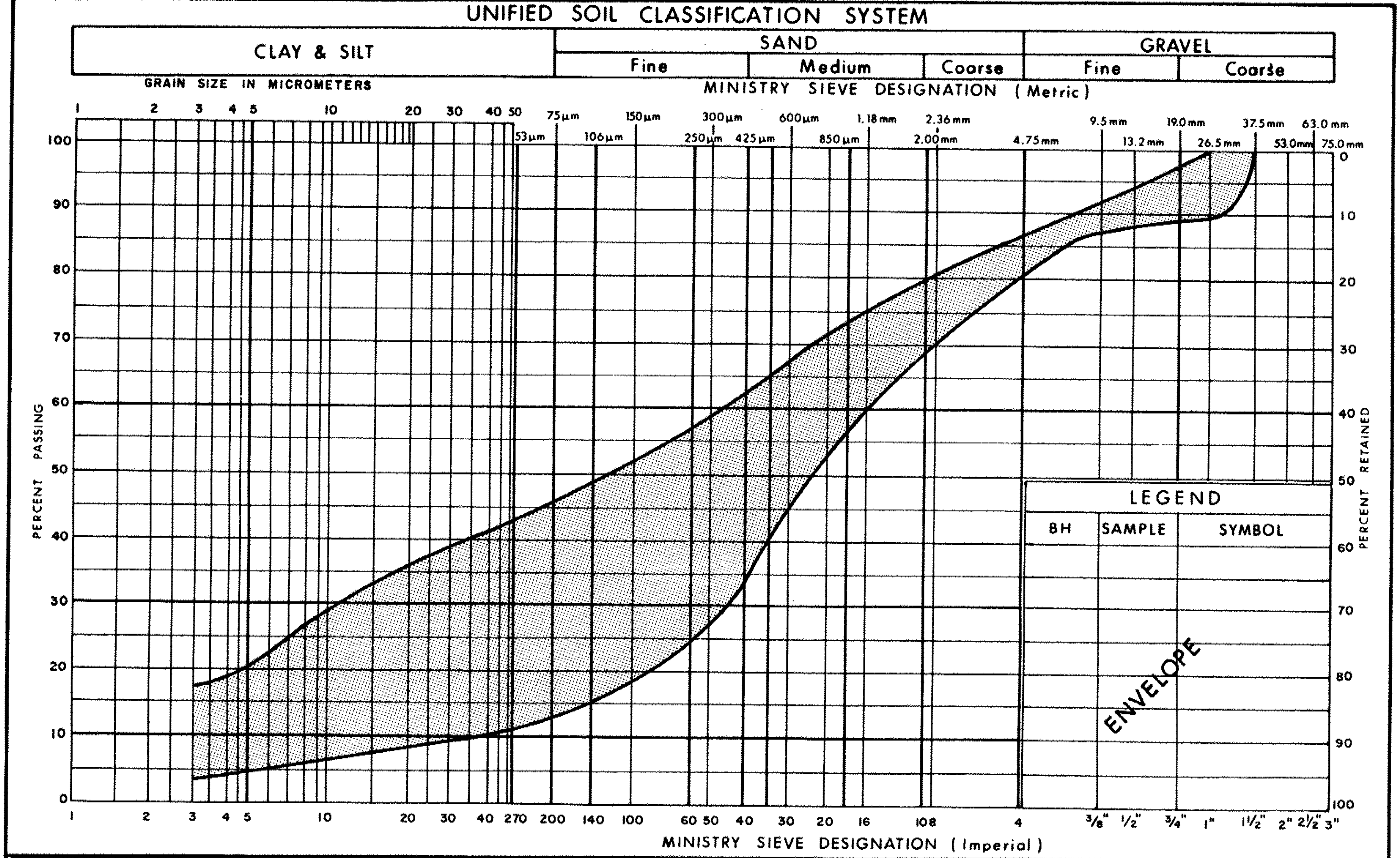
Ministry of
Transportation and
Communications
Ontario

# GRAIN SIZE DISTRIBUTION
# SILTY CLAY (FILL MATERIAL)

FIG No 1

W P 139 -86 -02

78 12 M



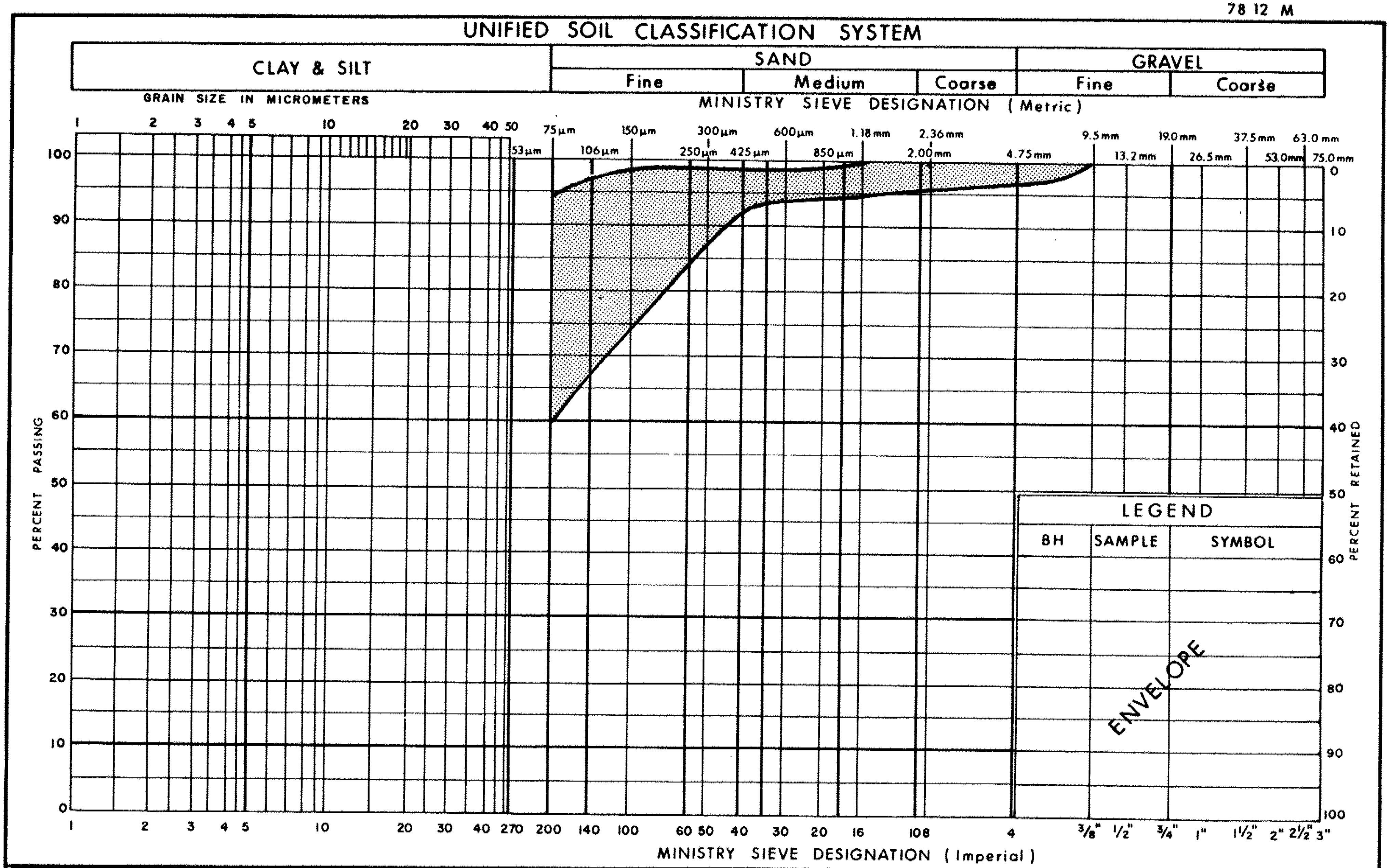
Ministry of
Transportation and
Communications
Ontario

GRAIN SIZE DISTRIBUTION
SAND (FILL MATERIAL)

FIG No 2

W P 139 - 86 - 02

78 12 M



Ministry of
Transportation and
Communications
Ontario

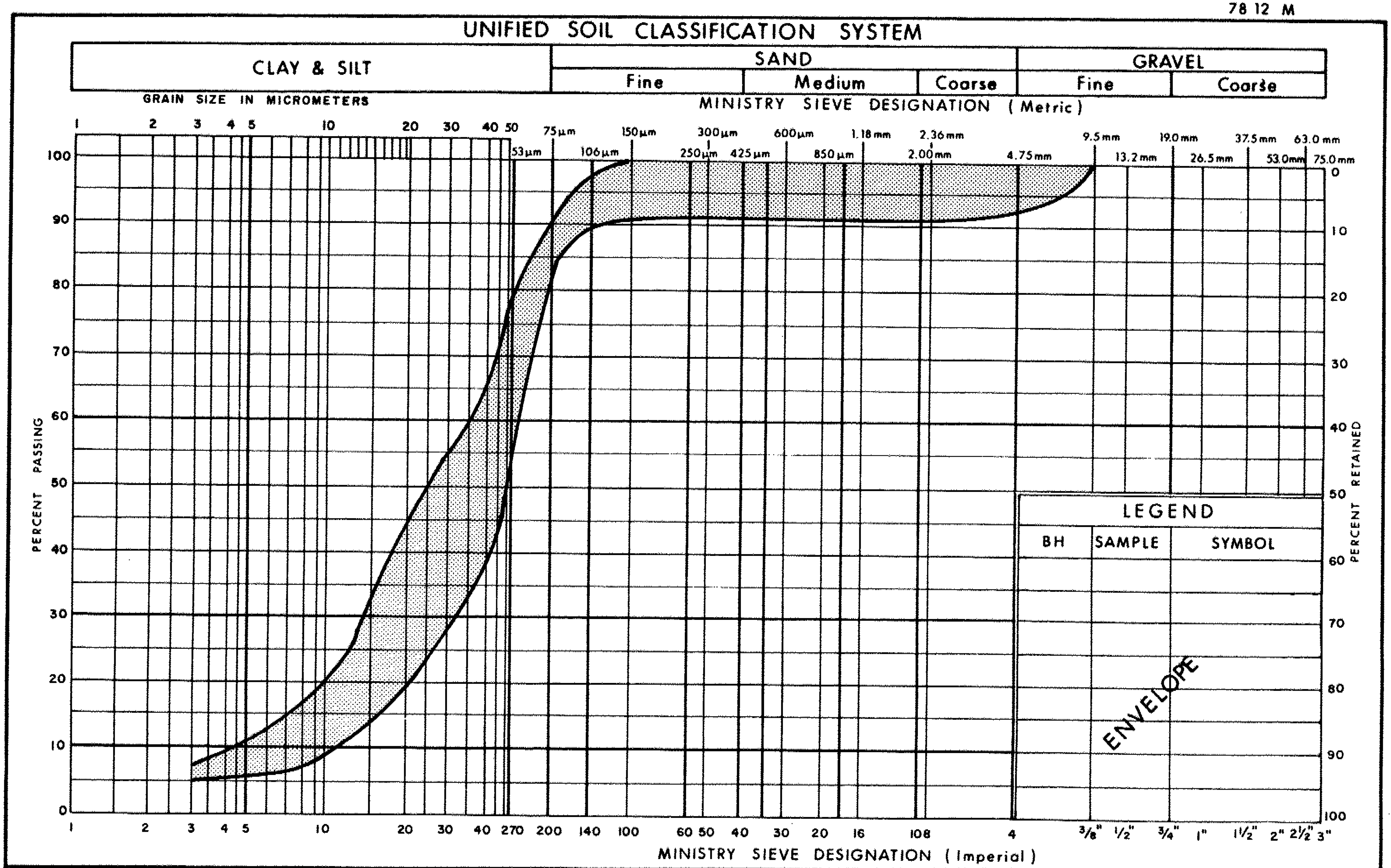
## GRAIN SIZE DISTRIBUTION

SILTY CLAY TRACE / SOME SAND, TRACE GRAVEL

FIG No 3

W P 139 - 86 - 02

78 12 M



Ministry of
Transportation and
Communications
Ontario

# GRAIN SIZE DISTRIBUTION
## SANDY SILT TO SILTY SAND

FIG No 4

W P 139 - 86 - 02

Ministry of Transportation and Communications
Ontario

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 1

METRIC

W P 139-86-02
DIST 2 HWY 401
DATUM Geodetic
LOCATION Co-ords. N 4 755 438.0; E 411 554.3
BOREHOLE TYPE Cont. Flight Auger (H.S.)
DATE 86 11 11
ORIGINATED BY DC
COMPILED BY PP
CHECKED BY

| SOIL PROFILE | | | SAMPLES | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV / DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | 'N' VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | PLASTIC LIMIT $W_p$ NATURAL MOISTURE CONTENT $W$ LIQUID LIMIT $W_L$ / WATER CONTENT (%) | UNIT WEIGHT $\gamma$ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
| 268.3 | Ground Level | | | | | | | | | | |
| 0.0 | Silty Clay<br>traces of sand<br>traces of organics<br>Very Stiff<br>to Hard | | | | | ▼ | 268 | | | | |
| | | | 1 | SS | 42 | | | | | | |
| | | | 2 | SS | 45 | | | | | | |
| | | | 3 | SS | 45 | | 266 | | | | |
| | | | 4 | SS | 29 | | | | | | |
| 264.0 | | | 5 | SS | 27 | | | | | | |
| 4.3 | End of Borehole | | | | | | | | | | |

+$^3$, x$^5$ : Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\diamond}} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 2

METRIC

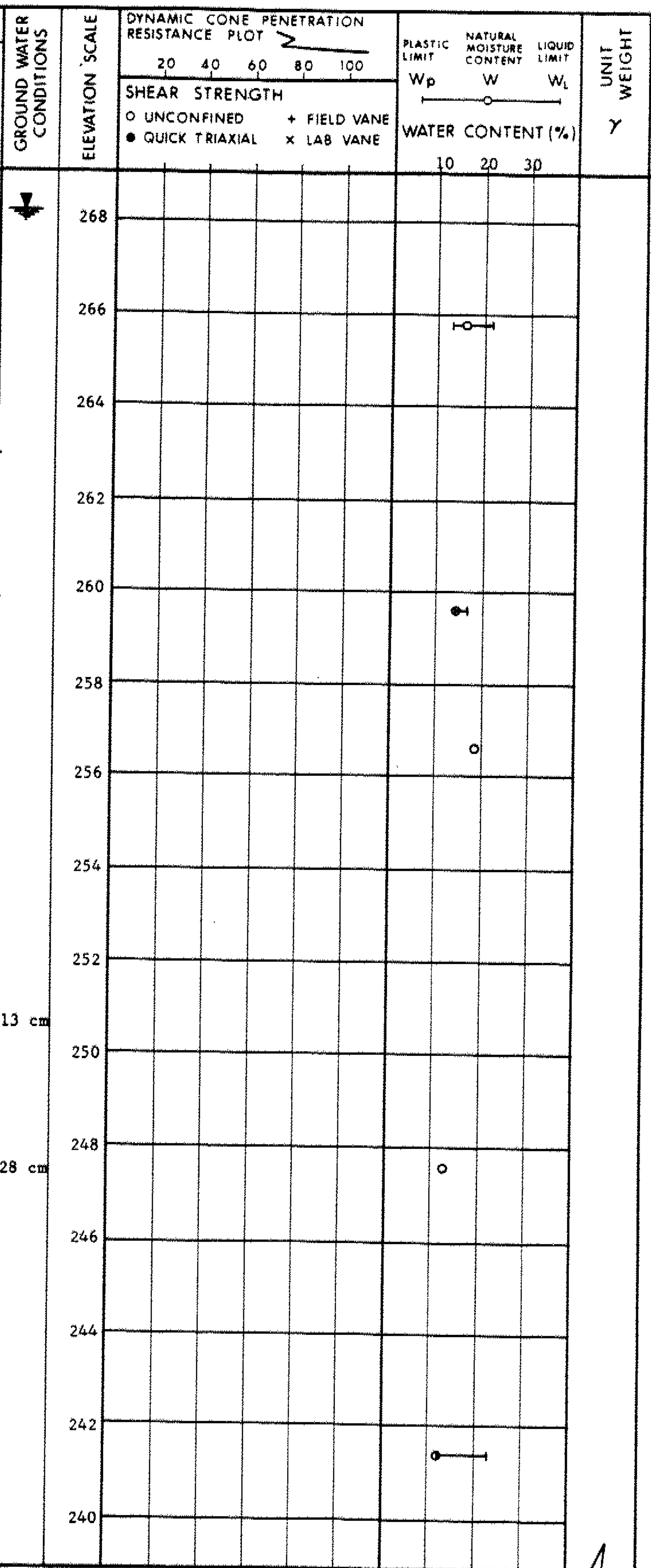
W P 139-86-02
LOCATION Co-ords: N 4 755 427.4; E 411 549.2
ORIGINATED BY DC
DIST 2 HWY 401
BOREHOLE TYPE Cont. Flight Auger (H.S.) & Washbore - BW Casing
COMPILED BY PP
DATUM Geodetic
DATE 86 11 11 - 86 11 13
CHECKED BY

**SOIL PROFILE**

| ELEV<br>DEPTH | DESCRIPTION |
|---|---|
| 269.0 | Ground Level |
| 0.0 | Silty Clay<br>traces of sand<br>traces of gravel<br>Very Stiff<br>to Hard |
| 258.3<br>10.7 | Silt<br>traces of sand<br>some clay<br>Dense |
| 255.3<br>13.7 | Silty Clay<br>traces of sand<br>Hard |
| 252.2<br>16.8 | Silt<br>trace/some sand<br>trace clay<br>Very<br>Dense |
| 247.5<br>21.5 | Sandy Silt<br>traces of clay<br>Very Dense |
| 243.1<br>25.9 | Silty Clay |

**SAMPLES**

| NUMBER | TYPE | 'N' VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
|---|---|---|---|
| 1 | SS | 86 | |
| 2 | SS | 27 | 0 7 (93) |
| 3 | SS | 23 | |
| 4 | SS | 20 | |
| 5 | SS | 24 | |
| 6 | SS | 17 | 0 5 (95) |
| 7 | SS | 31 | 0 5 80 15 |
| 8 | SS | 35 | |
| 9 | SS | 60/ 13 cm | |
| 10 | SS | 100/28 cm | 0 13 76 11 |
| 11 | SS | 50 | |
| 12 | SS | 84 | 2 20 (78) |



Continued

+3, x5 : Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\diamond}} 5$ (%) STRAIN AT FAILURE

Continued

Ontario Ministry of Transportation and Communications

# RECORD OF BOREHOLE No 2 (Cont'd) METRIC

W P 139-86-02
LOCATION Co-ords. N 4 755 427.4; E 411 549.2
ORIGINATED BY DC
DIST 2 HWY 401
BOREHOLE TYPE Cont. Flight Auger (H.S.) & Washbore - BW Casing
COMPILED BY PP
DATUM Geodetic
DATE 86 11 11 - 86 11 13
CHECKED BY

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100<br>SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | PLASTIC LIMIT $W_p$ NATURAL MOISTURE CONTENT W LIQUID LIMIT $W_L$<br>WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV / DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | 'N' VALUES | | | | | | |
| 238.8 | Continued | | | | | | | | | | |
| | | | 13 | SS | 15 | | | | | | |
| | | | | | | | 238 | | | | |
| | Some Sand | | | | | | | | | | |
| | traces of gravel | | | | | | | | | | |
| | | | | | | | 236 | | | | |
| | | | 14 | SS | 74 | | | | | | |
| | | | | | | | 234 | | | | |
| | Hard | | 15 | SS | 60 | | | | | | |
| | | | | | | | 232 | | | | |
| | | | | | | | 230 | | | | |
| 228.9 | | | 16 | SS | 30 | | | | ⊢○⊣ | | 4 16 (80) |
| 40.1 | End of Borehole | | | | | | | | | | |

OFFICE REPORT ON SOIL EXPLORATION

+³, ×⁵ : Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\diamond}} 5$ (%) STRAIN AT FAILURE

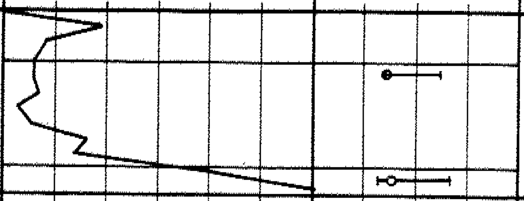
Ministry of Transportation and Communications
Ontario

# RECORD OF BOREHOLE No 3A

METRIC

W P 139-86-02
LOCATION Co-ords: N 4 755 507.3; E 411 554.8
ORIGINATED BY DC
DIST 2 HWY 401
BOREHOLE TYPE Cont. Flight Auger (S.S.)
COMPILED BY DC
DATUM Geodetic
DATE 86 12 03
CHECKED BY

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100<br>SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE | PLASTIC LIMIT Wp NATURAL MOISTURE CONTENT W LIQUID LIMIT WL<br>WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | 'N' VALUES | | | | | | |
| 269.0 | Ground Level | | | | | | | | | | |
| 0.0 | Silty Clay<br>with sand<br>traces of gravel<br>Soft<br>to<br>Hard | | | | | | | | | | |
| | | | 1 | SS | 5 | | 268 | | | | 7 38 (55) |
| | | | 2 | SS | 3 | | | | | | |
| | | | 3 | SS | 21 | | 266 | | | | |
| 265.5 | | | 4 | SS | 46 | | | | | | 3 11 (86) |
| 3.5 | End of Borehole | | | | | | | | | | |



OFFICE REPORT ON SOIL EXPLORATION

$+^3, \times^5$ : Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

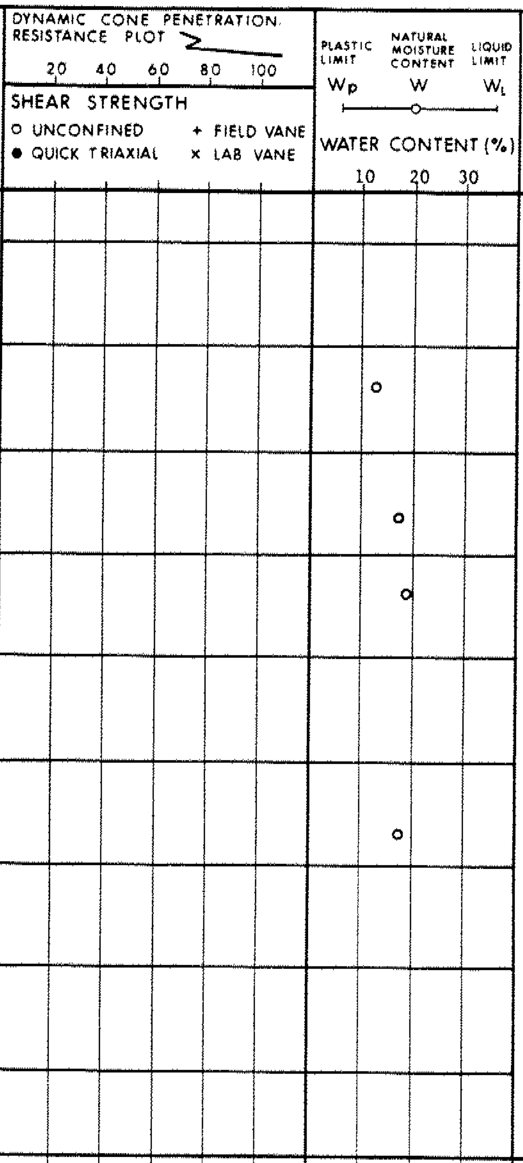
Ministry of Transportation and Communications
Ontario

# RECORD OF BOREHOLE No 3B

METRIC

W P 139-86-02
LOCATION Co-ords.N4 755 513.1; E 411 554.2
ORIGINATED BY DC
DIST 2 HWY 401
BOREHOLE TYPE Cont. Flight Auger (S.S.)
COMPILED BY PP
DATUM Geodetic
DATE 86 12 04
CHECKED BY

| SOIL PROFILE ELEV DEPTH | SOIL PROFILE DESCRIPTION | SAMPLES NUMBER | SAMPLES TYPE | SAMPLES 'N' VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 269.0 | Ground Level | | | | | | |
| 0.0 | Probably Silty Clay | | | | ▼ | 268<br>266 | |
| 265.3<br>3.7 | Silt Occasional Silty Clay Pockets traces of sand Very Dense | 5 | SS | 68 | | | 0 3 83 14 |
| | | 6 | SS | 55 | | 264 | |
| | | 7 | SS | 84 | | | 0 4 79 17 |
| 262.0<br>7.0 | Sandy Silt traces of clay traces of gravel Compact to Very Dense | | | | | 262 | |
| | | 8 | SS | 17 | | | 9 10 76 5 |
| | | 9 | SS | 31 | | 260<br>258 | |
| | | 10 | SS | 49 | | 256<br>254 | 0 16 79 5 |
| | | 11 | SS | 100/28 cm | | 252 | |
| 250.4 | | 12 | SS | 60/15 cm | | | |
| 18.6 | End of Borehole | | | | | | |



UNIT WEIGHT γ

+3, x5 : Numbers refer to Sensitivity

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

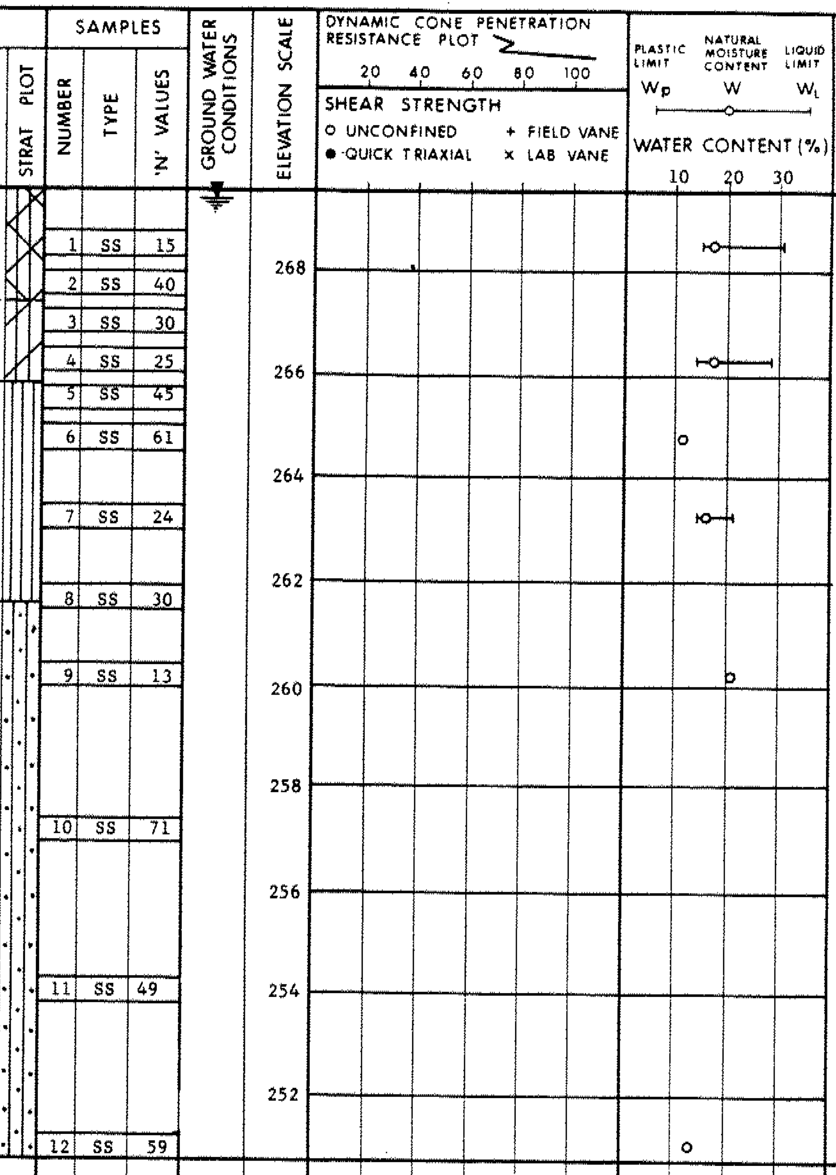
Ministry of Transportation and Communications
Ontario

# RECORD OF BOREHOLE No 4

METRIC

W P 139-86-02
DIST 2 HWY 401
DATUM Geodetic
LOCATION Co-ords. N 4 755 516.7; E 411 569.3
BOREHOLE TYPE Cont. Flight Auger (S.S.)
DATE 86 12 05
ORIGINATED BY DC
COMPILED BY DC
CHECKED BY

| SOIL PROFILE | | SAMPLES | | | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|---|
| ELEV / DEPTH | DESCRIPTION | NUMBER | TYPE | 'N' VALUES | GR | SA | SI | CL |
| 269.5 | Ground Level | | | | | | | |
| 0.0 | Silty Clay<br>traces of organics<br>traces of gravel<br>some sand (Fill Material)<br>Very Stiff to Hard | 1 | SS | 15 | 4 | 21 | (75) | |
| | | 2 | SS | 40 | | | | |
| | | 3 | SS | 30 | | | | |
| 265.8 | | 4 | SS | 25 | 1 | 11 | (88) | |
| 3.7 | Silt<br>traces of gravel<br>traces of sand<br>Occasional Silty Clay Seams<br>Compact to Very Dense | 5 | SS | 45 | | | | |
| | | 6 | SS | 61 | 8 | 4 | 79 | 9 |
| | | 7 | SS | 24 | 0 | 2 | (98) | |
| 261.6<br>7.9 | Sandy Silt to Silty Sand<br>traces of clay<br>Compact to Very Dense | 8 | SS | 30 | | | | |
| | | 9 | SS | 13 | | | | |
| | | 10 | SS | 71 | | | | |
| | | 11 | SS | 49 | | | | |
| 250.8 | | 12 | SS | 59 | | | | |
| 18.7 | End of Borehole | | | | | | | |



+3, x5 : Numbers refer to Sensitivity

20
15 -◊- 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

# RECORD OF BOREHOLE No 5

METRIC

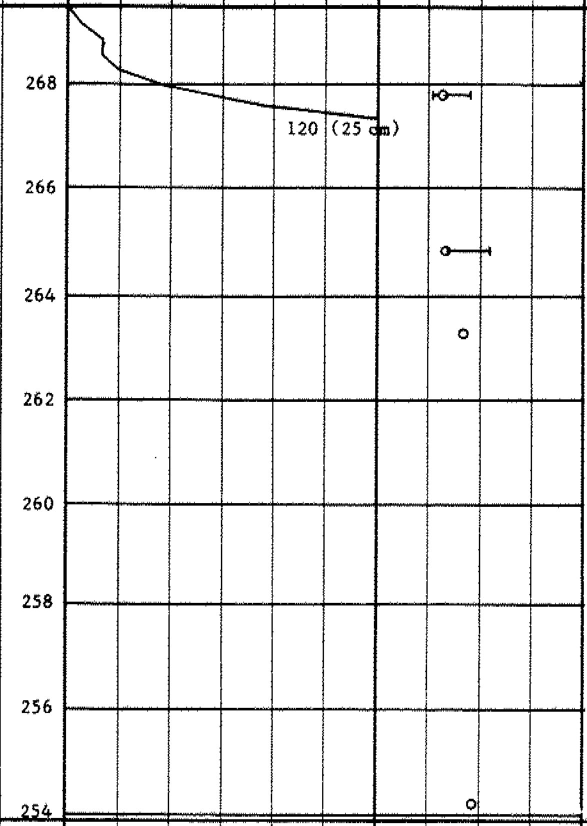
W P 139-86-02
LOCATION Co-ords. N 4 755 445.9; E 411 573.6
ORIGINATED BY DC
DIST 2 HWY 401
BOREHOLE TYPE Cont. Flight Auger (S.S.)
COMPILED BY PP
DATUM Geodetic
DATE 86 12 08
CHECKED BY

| SOIL PROFILE | | SAMPLES | | | | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | 'N' VALUES | GROUND WATER CONDITIONS | GR SA SI CL |
| 269.5 | Ground Level | | | | | |
| 0.0 | Silty Clay<br>trace/some sand<br>traces of gravel<br>Hard | | | | * | |
| | | 1 | SS | 32 | | |
| | | 2 | SS | 40 | | 2 38 (60) |
| | | 3 | SS | 41 | | |
| | | 4 | SS | 25 | | |
| | | 5 | SS | 54 | | 0 2 (98) |
| 263.1 | | 6 | SS | 55 | | |
| 6.4 | Sandy Silt<br>to<br>Silty Sand<br>traces of clay<br>Compact<br>to<br>Dense | | | | | |
| | | 7 | SS | 19 | | |
| | | 3 | SS | 50 | | |
| 253.8 | | 9 | SS | 45 | | 7 38 (55) |
| 15.7 | End of Borehole<br>* Groundwater Level not observed | | | | | |

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100
SHEAR STRENGTH
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT $W_p$ NATURAL MOISTURE CONTENT $W$ LIQUID LIMIT $W_L$
WATER CONTENT (%) 10 20 30

UNIT WEIGHT $\gamma$



$^{+3}, ^{\times 5}$ : Numbers refer to Sensitivity

20
15 ◇ 5 (%) STRAIN AT FAILURE
10

Ministry of Transportation and Communications
Ontario

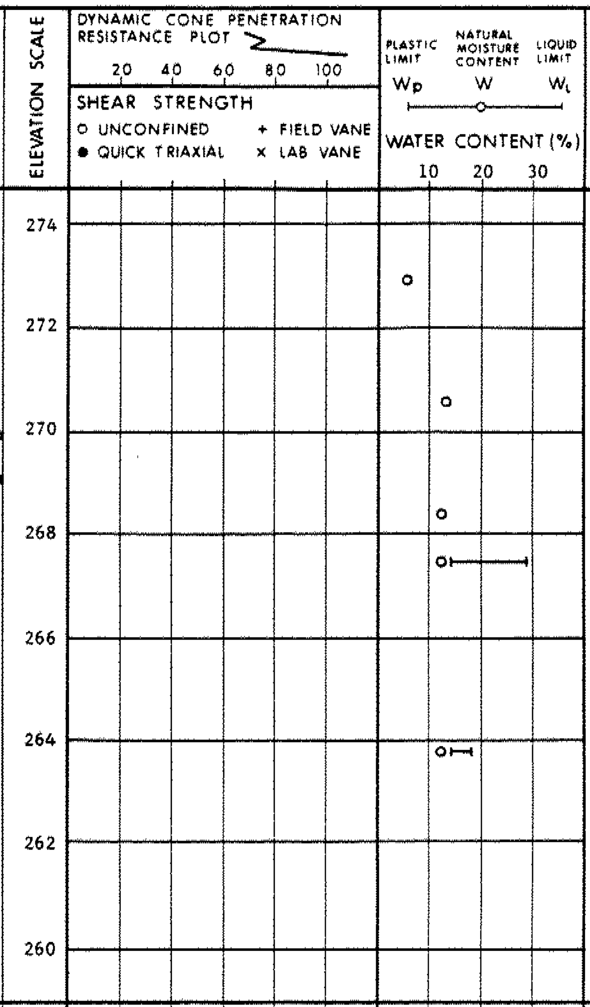
OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 101

METRIC

W P 139-86-02
DIST 2 HWY 401
DATUM Geodetic
LOCATION Co-ords: N 4 755 456.3; E 411 548.6
BOREHOLE TYPE Washbore - NX Casing
DATE 86 11 18 - 86 11 20
ORIGINATED BY DM
COMPILED BY PP
CHECKED BY

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | SAMPLES NUMBER | TYPE | 'N' VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| 274.7 | Ground Level | | | | | |
| 0.0 | Sand<br>Some Gravel<br>Occasional Cobbles<br>traces of silt<br>traces of clay<br>Loose to<br>Very Dense<br>(Fill Material) | 1 | SS | 28 | | |
| | | 2 | SS | 11 | | 19 67 10 4 |
| | | 3 | SS | 7 | | |
| | | 4 | SS | 7 | | |
| | | 5 | SS | 14 | | 14 41 29 16 |
| | | 6 | SS | 60/7.5 cm | | |
| | | 7 | SS | 60/10 cm | ▼ | |
| 268.0 | | 8 | SS | 45 | | 13 73 10 4 |
| 6.7 | Silty Clay<br>trace/with sand<br>traces of gravel<br>Occasional Silt Seams<br>and Layers<br>Hard | 9 | SS | 44 | | 1 30 (69) |
| | | 10 | SS | 67 | | |
| | | 11 | SS | 32 | | |
| | | 12 | SS | 60 | | 0 5 (95) |
| | | 13 | SS | 46 | | |
| 259.0 | | 14 | SS | 46 | | |
| 15.7 | End of Borehole | | | | | |



+3, ×5 : Numbers refer to Sensitivity

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation and Communications
Ontario

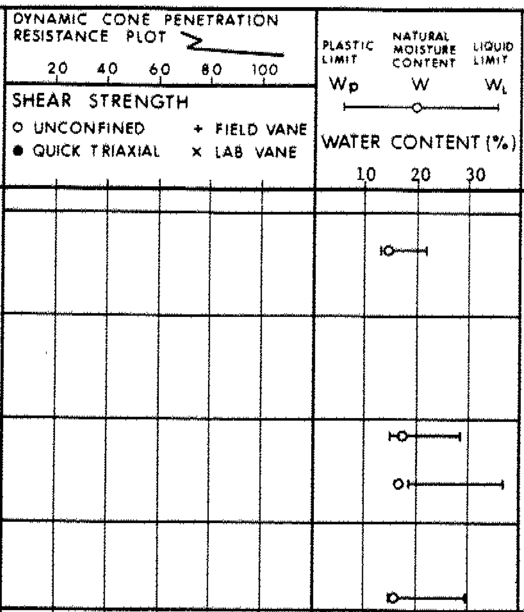
OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 102

METRIC

W P 139-86-02
DIST 2 HWY 401
DATUM Geodetic
LOCATION Co-ords: N 4 755 503.6; E 411 577.0
BOREHOLE TYPE Washbore - NX Casing
DATE 86 11 21 - 86 11 24
ORIGINATED BY DM
COMPILED BY DM
CHECKED BY

| SOIL PROFILE | | | SAMPLES | | | | | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---|---|---|---|---|---|---|---|---|
| ELEV / DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | 'N' VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | GR SA SI CL |
| 274.5 | Ground Level | | | | | | | |
| 0.0 | Silty Clay<br>some/with sand<br>trace/some gravel<br>Firm to<br>Very Stiff<br>(Fill Material) | | | | | ▼* | 274 | |
| | | | 1 | SS | 4 | | | 13 28 (59) |
| | | | 2 | SS | 4 | | | |
| | | | 3 | SS | 10 | | 272 | |
| | | | 4 | SS | 6 | | | |
| | | | 5 | SS | 24 | | 270 | |
| 269.3 | | | 6 | SS | 22 | | | 3 35 (62) |
| 5.2 | Silty Clay<br>trace/some sand<br>traces of gravel<br>Hard | | 7 | SS | 41 | | | 1 12 (87) |
| | | | 8 | SS | 50 | | 268 | |
| | | | 9 | SS | 43 | | | |
| 266.4 | | | 10 | SS | 25 | | | 6 16 (78) |
| 8.1 | End of Borehole | | | | | | | |

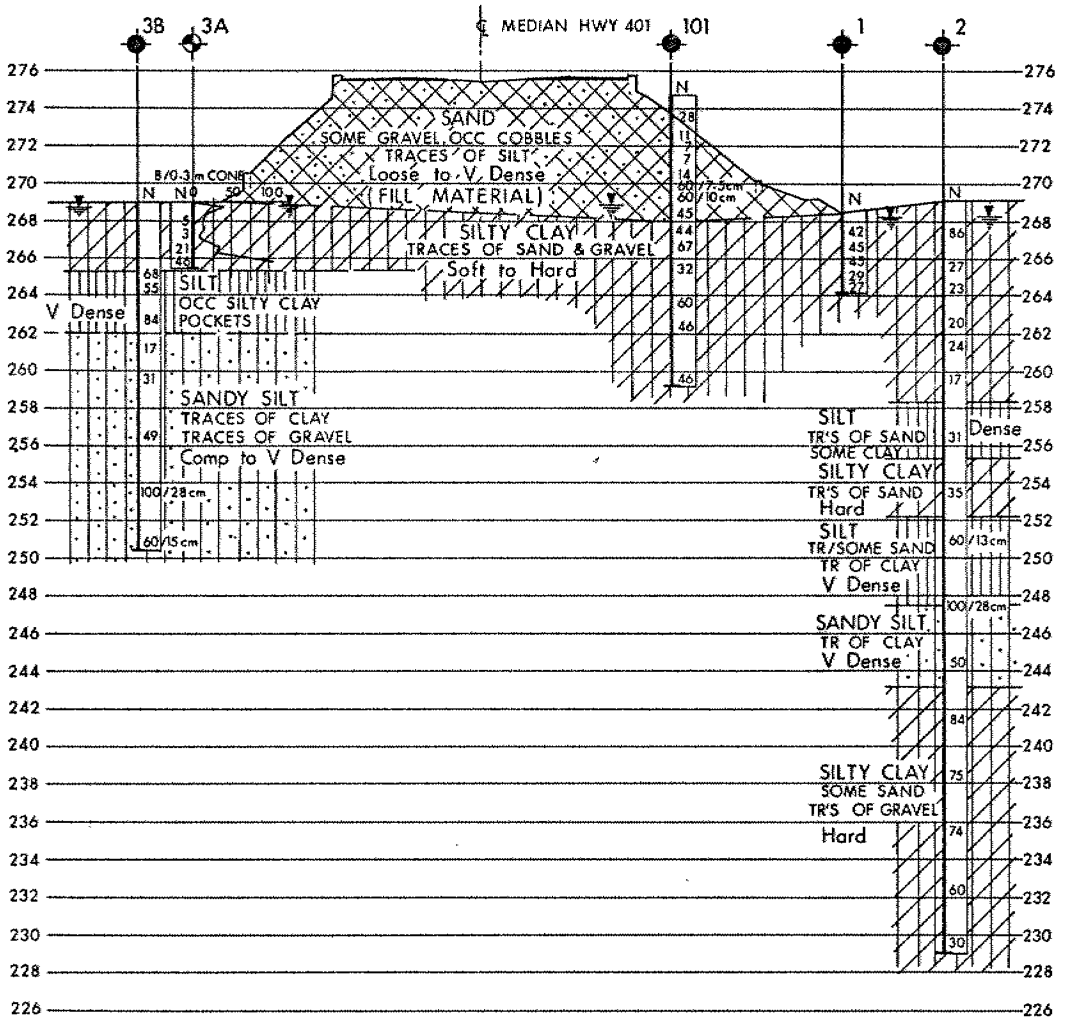


* Water Level was observed to be 0.6 m below ground level, one day after the removal of casings.

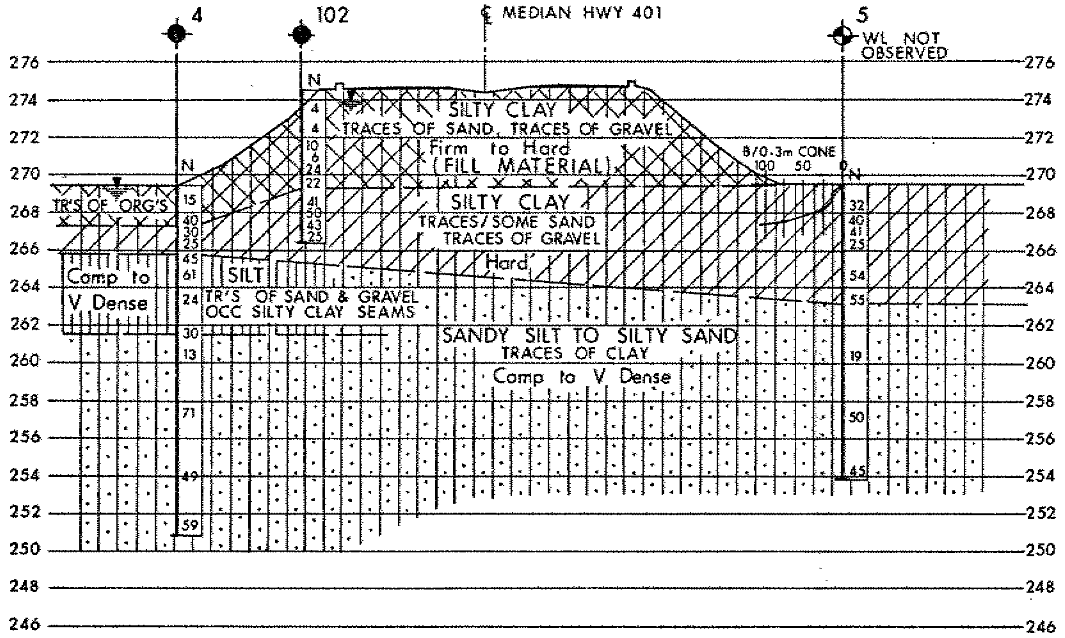
$+^3$, $\times^5$ : Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\diamond}} 5$ (%) STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO PH-D-207 82 04



A-A



B-B

## SECTIONS

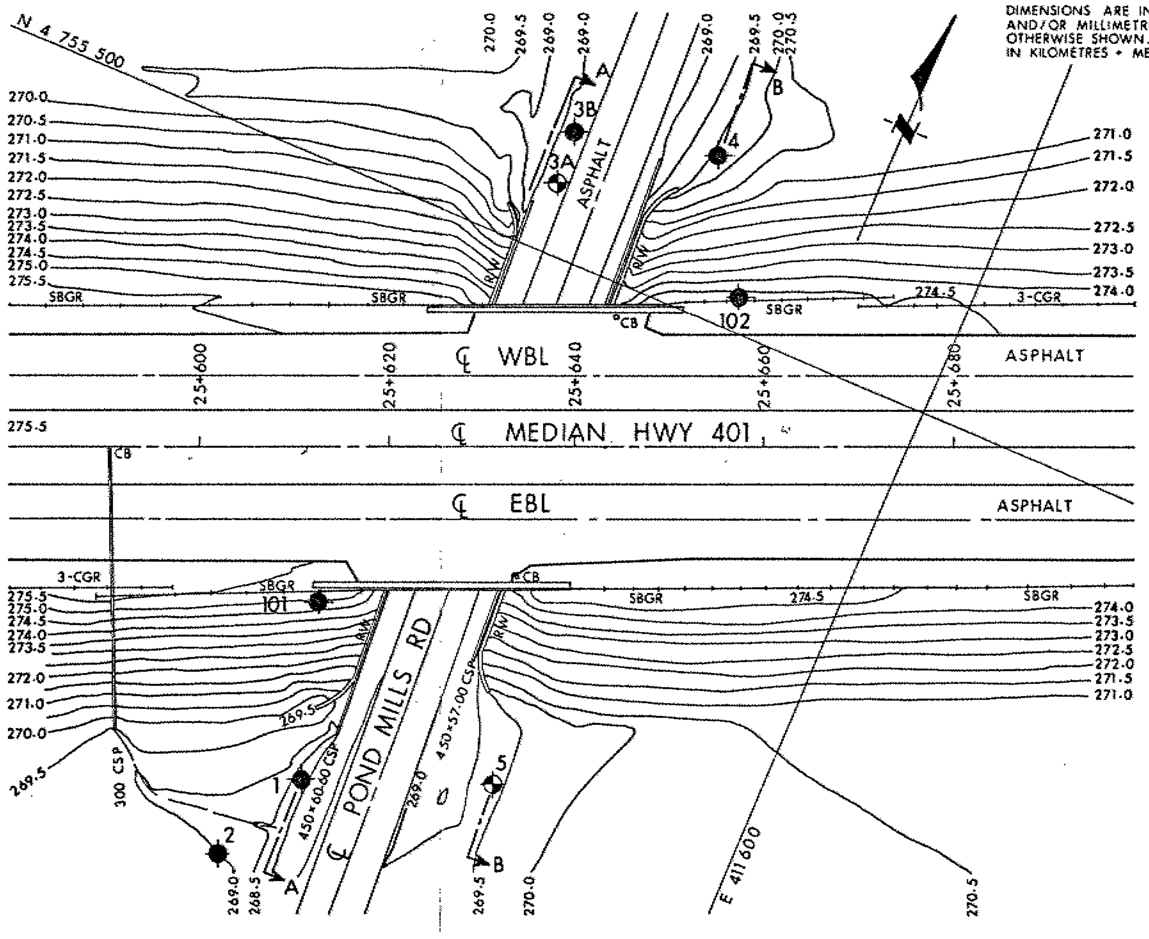
SCALE

8m 4 0 8m Hor

4m 2 0 4m Vert

# METRIC

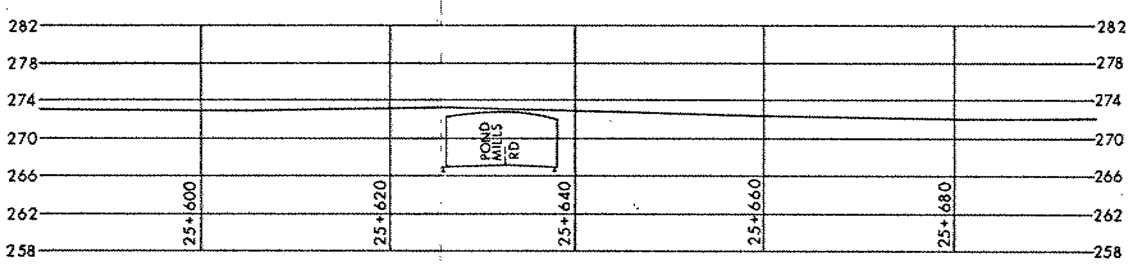
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



PLAN

SCALE

8m 4 0 8m



℄ PROFILE HWY 401

SCALE

8m 4 0 8m

REF No E-10030-1 84 02

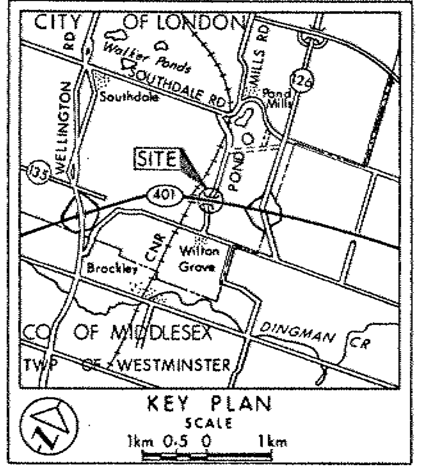
CONT No

WP No 139-86-02

SHEET

POND MILLS ROAD

BORE HOLE LOCATIONS & SOIL STRATA



KEY PLAN

SCALE

1km 0.5 0 1km

## LEGEND

| | |
|---|---|
| ● | Bore Hole |
| ⊕ | Dynamic Cone Penetration Test (Cone) |
| ◆ | Bore Hole & Cone |
| N | Blows/0·3m (Std Pen Test, 475 J/blow) |
| CONE | Blows/0·3m (60° Cone, 475 J/blow) |
| ⊻ | W L at time of investigation 86 11 |

| No | ELEVATION | CO-ORDINATES NORTH | CO-ORDINATES EAST |
|---|---|---|---|
| 1 | 268·3 | 4 755 438·0 | 411 554·3 |
| 2 | 269·0 | 4 755 427·4 | 411 549·2 |
| 3A | 269·0 | 4 755 507·3 | 411 554·8 |
| 3B | 269·0 | 4 755 513·1 | 411 554·2 |
| 4 | 269·5 | 4 755 516·7 | 411 569·3 |
| 5 | 269·5 | 4 755 445·9 | 411 573·6 |
| 101 | 274·7 | 4 755 456·3 | 411 548·6 |
| 102 | 274·5 | 4 755 503·6 | 411 577·0 |

=NOTE=

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

| REV. | DATE | BY | DESCRIPTION |
|---|---|---|---|

Geocres No 40114-111

| | | | |
|---|---|---|---|
| HWY No 401 | | | DIST 2 |
| SUBM'D PP | CHECKED | DATE 87 03 26 | SITE 19-372 |
| DRAWN DT | CHECKED | APPROVED | DWG 1398602-A |

# 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, 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 $cm^2$ pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ($Q_t$), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

## III. SOIL DESCRIPTION

### (a) Cohesionless Soils

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

### (b) Cohesive Soils

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

## IV. SOIL TESTS

| | |
|---|---|
| w | water content |
| $w_p$ | plastic limit |
| $w_l$ | liquid limit |
| C | consolidation (oedometer) test |
| CHEM | chemical analysis (refer to text) |
| CID | consolidated isotropically drained triaxial test[1] |
| CIU | consolidated isotropically undrained triaxial test with porewater pressure measurement[1] |
| $D_R$ | relative density (specific gravity, $G_s$) |
| DS | direct shear test |
| M | sieve analysis for particle size |
| MH | combined sieve and hydrometer (H) analysis |
| MPC | Modified Proctor compaction test |
| SPC | Standard Proctor compaction test |
| OC | organic content test |
| $SO_4$ | concentration of water-soluble sulphates |
| UC | unconfined compression test |
| UU | unconsolidated undrained triaxial test |
| V | field vane (LV-laboratory vane test) |
| $\gamma$ | 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 $= (e_{max} - e) / (e_{max} - e_{min})$ (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 13-101

1 OF 2

METRIC

PROJECT 12-1132-0076
W.P. 3030-11-00
LOCATION N 4755728.8 , E 411548.9
ORIGINATED BY BT
DIST ____ HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF/LMK
DATUM GEODETIC
DATE February 6 - 7, 2013
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/m³

| ELEV<br>DEPTH | SOIL PROFILE<br>DESCRIPTION | SAMPLES<br>NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
|---|---|---|---|---|---|---|
| 269.72 | GROUND SURFACE | | | | | |
| 0.00 | TOPSOIL, silty<br>Brown | | | | | |
| 0.30 | FILL, silty sand, trace topsoil<br>Brown | | | | | |
| 268.96<br>0.76 | CLAYEY SILT TILL, trace sand,<br>trace gravel<br>Very stiff<br>Brown | 1 | SS | 18 | | |
| | | 2 | SS | 21 | Granular Bentonite | |
| | | 3 | SS | 25 | | |
| | | 4 | SS | 23 | | 1 7 50 42 |
| | | 5 | SS | 15 | | |
| 265.45<br>4.27 | SANDY SILT, trace clay<br>Compact to dense<br>Grey | 6 | SS | 31 | | |
| | | 7 | SS | 28 | Backfill | |
| 264.33<br>5.39 | SILT, some sand, trace clay, with<br>clayey silt layers<br>Compact to dense<br>Grey | 8 | SS | 31 | | |
| | | 9 | SS | 20 | Standpipe | |
| 262.80<br>6.92 | SILT, trace sand, some clay<br>Very loose to compact<br>Grey | 10 | SS | 19 | | 0 9 74 17 |
| | | 11 | SS | WR | | |
| 259.97<br>9.75 | SILT, trace clay, with clayey silt<br>layers<br>Dense<br>Grey | 12 | SS | 30 | | 0 1 91 8 |
| 257.65<br>12.07 | CLAYEY SILT, with silt seams and<br>layers<br>Stiff to very stiff<br>Grey | 13 | SS | 19 | | |
| | | 14 | SS | 25 | | 0 1 74 25 |

Continued Next Page

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

---

Golder Associates
London, Ontario

## RECORD OF BOREHOLE No 13-101

2 OF 2

METRIC

PROJECT 12-1132-0076
W.P. 3030-11-00
LOCATION N 4755728.8 , E 411548.9
ORIGINATED BY BT
DIST ____ HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF/LMK
DATUM GEODETIC
DATE February 6 - 7, 2013
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/m³

| ELEV<br>DEPTH | SOIL PROFILE<br>DESCRIPTION | SAMPLES<br>NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
|---|---|---|---|---|---|---|
| | CLAYEY SILT, with silt seams and<br>layers<br>Stiff to very stiff<br>Grey | 15 | SS | 16 | | |
| | | 16 | SS | 13 | | |
| 252.04<br>17.68 | SILT, trace clay, trace to some sand<br>Very dense<br>Grey | 17 | SS | 101/<br>200mm | | 0 10 81 9 |
| | | 18 | SS | 100 | Granular Bentonite<br>Filter Sand<br>Piezometer | |
| 248.99<br>20.73 | SAND, fine, trace silt, with silt layers<br>Very dense<br>Grey | 19 | SS | 102/<br>225mm | Caved Material | |
| 248.29<br>21.43 | END OF BOREHOLE | | | | | |

Groundwater encountered at about elev. 262.8m during drilling on February 6, 2013.

Water level measured in standpipe at elev. 266.52m on February 11, 2013.

Water level measured in standpipe at elev. 266.62m on March 8, 2013.

Water level measured in piezometer at elev. 266.52m on March 8, 2013.

Water level measured in standpipe at elev. 266.77m on April 3, 2013.

Water level measured in piezometer at elev. 266.72m on April 3, 2013.

Water level measured in standpipe at elev. 267.05m on June 5, 2013.

Water level measured in piezometer at elev. 266.62m on June 5, 2013.

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

Golder Associates
London, Ontario

## RECORD OF BOREHOLE No 13-102 1 OF 2 METRIC

| | | | |
|---|---|---|---|
| PROJECT 12-1132-0076 | | | |
| W.P. 3030-11-00 | LOCATION N 4755673.4 , E 411594.4 | | ORIGINATED BY BT |
| DIST | HWY 401 | BOREHOLE TYPE POWER AUGER, HOLLOW STEM | COMPILED BY WDF/LMK |
| DATUM GEODETIC | DATE February 14 - 19, 2013 | | CHECKED BY |

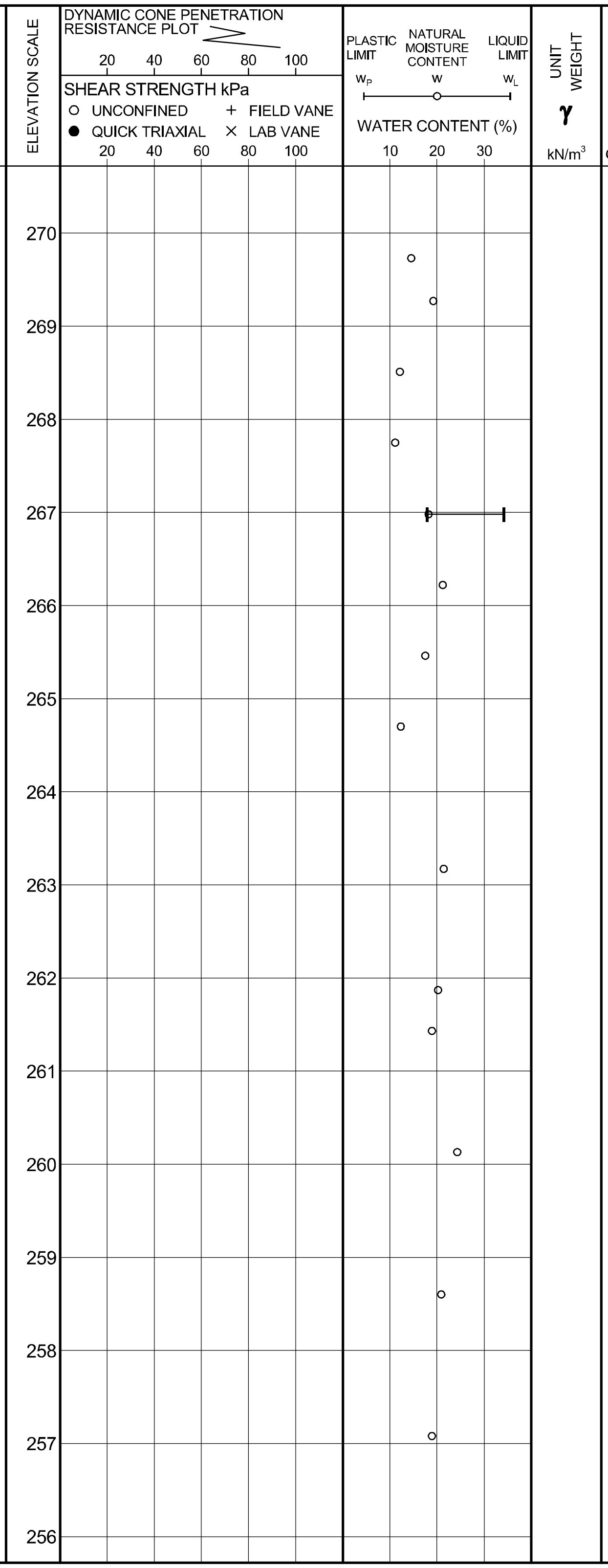
SOIL PROFILE

| ELEV / DEPTH | DESCRIPTION |
|---|---|
| 270.72 | GROUND SURFACE |
| 0.00 | TOPSOIL, silty, trace gravel<br>Brown |
| 270.26 / 0.46 | FILL, silt, some sand, trace topsoil<br>Compact<br>Brown |
| 269.50 / 1.22 | FILL, clayey silt, trace to some sand, trace gravel, with cobbles<br>Loose to compact<br>Brown |
| 267.37 / 3.35 | CLAYEY SILT TILL, trace sand, trace gravel<br>Very stiff<br>Brown |
| 266.61 / 4.11 | SANDY SILT,<br>Compact<br>Brown becoming grey at about elev. 266.1m |
| 265.84 / 4.88 | CLAYEY SILT TILL, trace sand, trace gravel<br>Stiff<br>Grey |
| 265.08 / 5.64 | SILT, trace to some sand, trace clay, with clayey silt layers<br>Compact<br>Grey |
| 262.49 / 8.23 | SANDY SILT<br>Dense<br>Grey |
| 261.55 / 9.17 | SILT, trace sand, trace clay<br>Dense<br>Grey |
| 260.81 / 9.91 | SILT, some sand, with silt layers<br>Compact<br>Grey |
| 256.39 / 14.33 | CLAYEY SILT, trace sand, with silt layers<br>Very stiff to hard<br>Grey |

Continued Next Page

SAMPLES

| NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|
| 1 | SS | 15 | ∇ | | | | |
| 2 | SS | 8 | | | | | |
| 3 | SS | 18 | | | | | |
| 4 | SS | 25 | | | | | |
| 5 | SS | 19 | | 0 | 8 | 49 | 43 |
| 6 | SS | 20 | | | | | |
| 7 | SS | 14 | | | | | |
| 8 | SS | 25 | | | | | |
| 9 | SS | 13 | | | | | |
| 10 | SS | 31 | | 0 | 32 | 63 | 5 |
| 11 | SS | 23 | | | | | |
| 12 | SS | 21 | | 0 | 14 | 80 | 6 |
| 13 | SS | 27 | | | | | |



REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

---

Golder Associates
London, Ontario

## RECORD OF BOREHOLE No 13-102 2 OF 2 METRIC

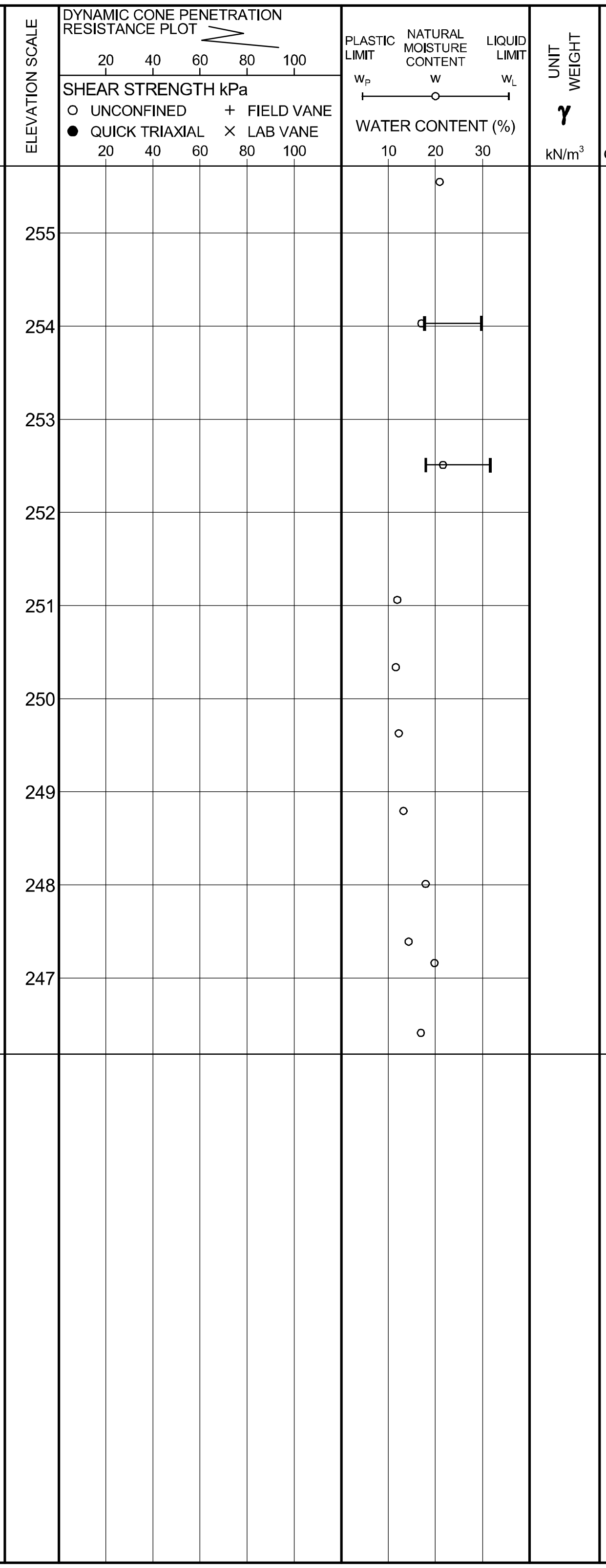
| | | | |
|---|---|---|---|
| PROJECT 12-1132-0076 | | | |
| W.P. 3030-11-00 | LOCATION N 4755673.4 , E 411594.4 | | ORIGINATED BY BT |
| DIST | HWY 401 | BOREHOLE TYPE POWER AUGER, HOLLOW STEM | COMPILED BY WDF/LMK |
| DATUM GEODETIC | DATE February 14 - 19, 2013 | | CHECKED BY |

SOIL PROFILE

| ELEV / DEPTH | DESCRIPTION |
|---|---|
| | CLAYEY SILT, trace sand, with silt layers<br>Very stiff to hard<br>Grey |
| 251.67 / 19.05 | SILT, trace to some sand, trace gravel<br>Very dense<br>Grey |
| 247.31 / 23.41 | SAND, fine, some silt<br>Very dense<br>Grey |
| 246.79 / 23.93 | CLAYEY SILT, trace sand, trace gravel<br>Very stiff<br>Grey |
| 246.18 / 24.54 | END OF BOREHOLE<br><br>Groundwater encountered at about elev. 269.5m during drilling on February 14, 2013. |

SAMPLES

| NUMBER | TYPE | "N" VALUES | GR | SA | SI | CL |
|---|---|---|---|---|---|---|
| 14 | SS | 42 | | | | |
| 15 | SS | 26 | 0 | 1 | 51 | 48 |
| 16 | SS | 20 | 0 | 1 | 48 | 51 |
| 17 | SS | 132 | | | | |
| 18 | SS | 100/ 225mm | | | | |
| 19 | SS | 100/ 125mm | 0 | 12 | 77 | 11 |
| 20 | SS | 100/ 275mm | | | | |
| 21 | SS | 103 | | | | |
| 22 | SS | 100/ 250mm | | | | |
| 23 | SS | 20 | | | | |



REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Golder Associates
London, Ontario

# RECORD OF BOREHOLE No 13-103

1 OF 1 **METRIC**

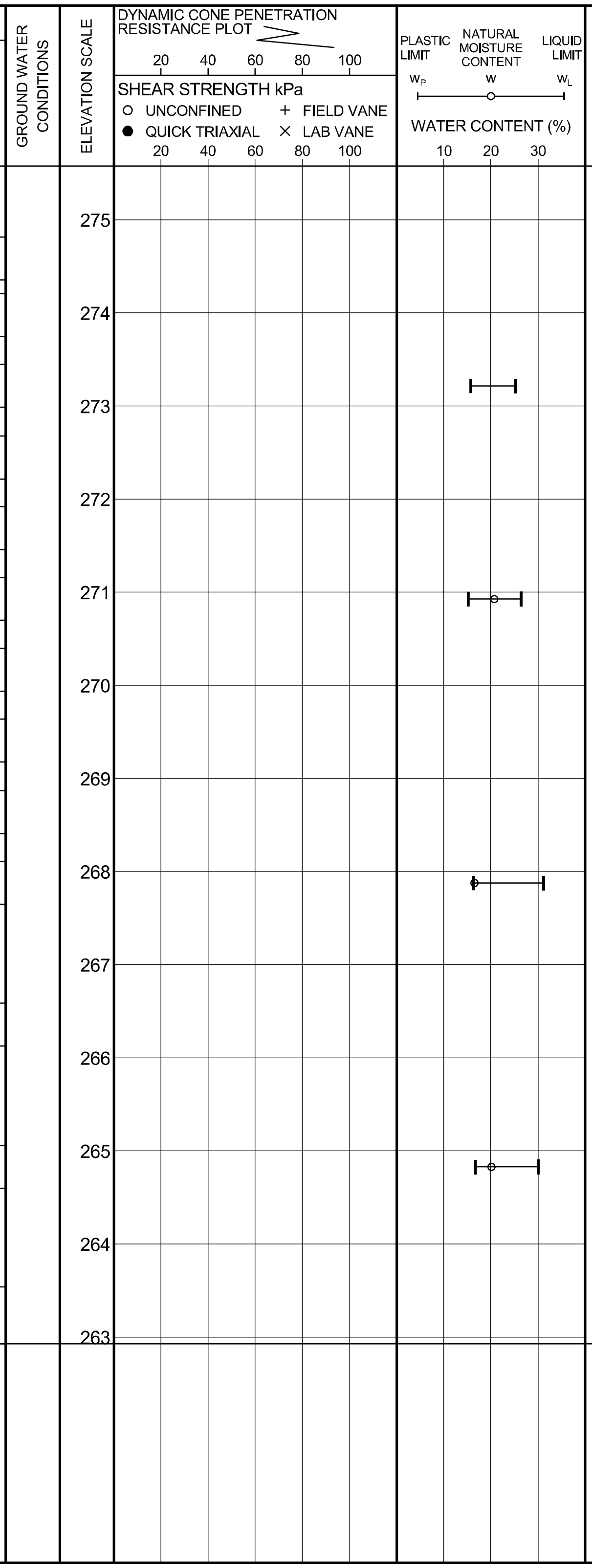
PROJECT 12-1132-0076
W.P. 3030-11-00
LOCATION N 4755703.8 , E 411539.8
ORIGINATED BY MA
DIST HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY AMG/LMK
DATUM GEODETIC
DATE May 14, 2013
CHECKED BY

**SOIL PROFILE**

| ELEV / DEPTH | DESCRIPTION |
|---|---|
| 275.58 | PAVEMENT SURFACE |
| 0.00 | ASPHALT |
| 0.12<br>0.30 | FILL, sand and gravel, trace silt, crushed<br>Brown |
| 274.60<br>0.98 | FILL, sand and gravel, trace silt, cobbles<br>Compact<br>Brown |
| | FILL, clayey silt, trace to some sand, trace gravel, trace to some topsoil, with silt seams and layers<br>Firm to hard<br>Brown |
| 270.00<br>5.58 | FILL, topsoil, silty, trace sand, trace gravel<br>Very stiff<br>Black |
| 5.79<br>269.27 | FILL, clayey silt, some sand, some topsoil, trace gravel<br>Very stiff<br>Brown |
| 6.31<br>6.55 | FILL, topsoil, silty, some sand<br>Very stiff<br>Black |
| | CLAYEY SILT TILL, trace to some sand, trace to some gravel<br>Very stiff to hard<br>Brown becoming grey at about elev. 266.4m |
| 264.00<br>11.58 | SILT, trace clay<br>Compact<br>Grey |
| 262.93<br>12.65 | END OF BOREHOLE<br><br>Borehole dry during drilling on May 14, 2013. |

**SAMPLES**

| NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|
| 1 | SS | 12 | |
| 2 | SS | 65 | |
| 3 | SS | 7 | 0 25 45 30 |
| 4 | SS | 6 | |
| 5 | SS | 5 | |
| 6 | SS | 12 | 0 17 54 29 |
| 7 | SS | 18 | |
| 8 | SS | 20 | |
| 9 | SS | 28 | |
| 10 | SS | 43 | 7 11 44 38 |
| 11 | SS | 28 | |
| 12 | SS | 19 | 20 11 37 32 |
| 13 | SS | 17 | |



UNIT WEIGHT γ kN/m³

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

---

Golder Associates
London, Ontario

# RECORD OF BOREHOLE No 13-104

1 OF 1 **METRIC**

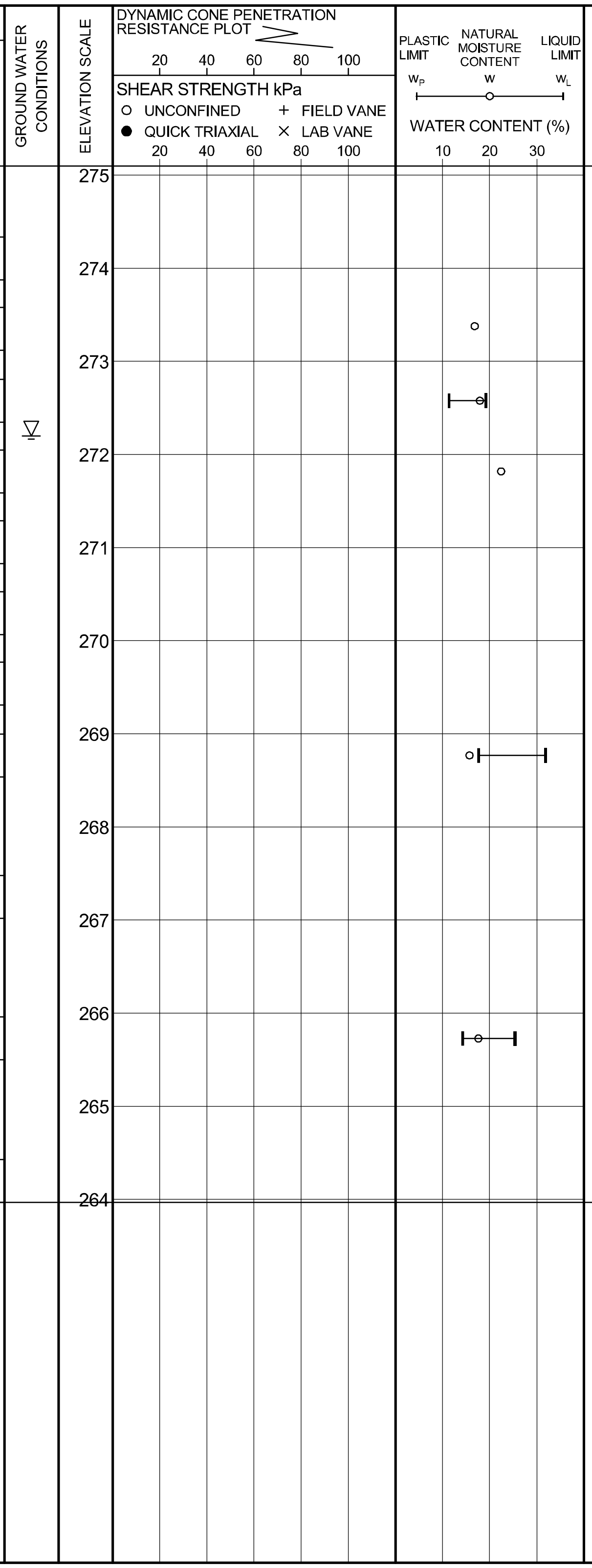
PROJECT 12-1132-0076
W.P. 3030-11-00
LOCATION N 4755694.3 , E 411594.5
ORIGINATED BY MA
DIST HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY AMG/LMK
DATUM GEODETIC
DATE May 15, 2013
CHECKED BY

**SOIL PROFILE**

| ELEV / DEPTH | DESCRIPTION |
|---|---|
| 275.10 | PAVEMENT SURFACE |
| | ASPHALT |
| 0.11<br>0.34 | FILL, sand and gravel, trace silt, crushed<br>Brown |
| 274.09<br>1.01 | FILL, sand and gravel, trace silt, cobbles<br>Compact<br>Brown |
| 273.73<br>1.37 | FILL, clayey silt, trace sand, trace gravel, trace topsoil<br>Stiff<br>Brown |
| 273.18<br>1.92 | FILL, sandy clayey silt, some topsoil<br>Firm<br>Brown |
| 2.13 | FILL, silty fine sand<br>Loose<br>Brown |
| 272.20<br>2.90 | FILL, clayey silt, with silt and sand seams and layers<br>Firm<br>Brown |
| 271.08<br>4.02 | FILL, fine sand, some silt, with clayey silt layers<br>Loose<br>Brown |
| | FILL, clayey silt, trace sand, trace topsoil<br>Firm to very stiff<br>Brown |
| 269.92<br>5.18 | CLAYEY SILT TILL, some sand, trace gravel<br>Very stiff to hard<br>Brown becoming grey at about elev. 268.1m |
| 267.18<br>7.92 | SILT, trace clay, with clayey silt layers<br>Compact<br>Grey |
| 266.26<br>8.84 | CLAYEY SILT TILL, some sand, trace gravel<br>Very stiff<br>Grey |
| 265.04<br>10.06 | SILT, with clayey silt layers<br>Dense<br>Grey |
| 263.97<br>11.13 | END OF BOREHOLE<br><br>Groundwater encountered at about elev. 272.2m during drilling on May 15, 2013. |

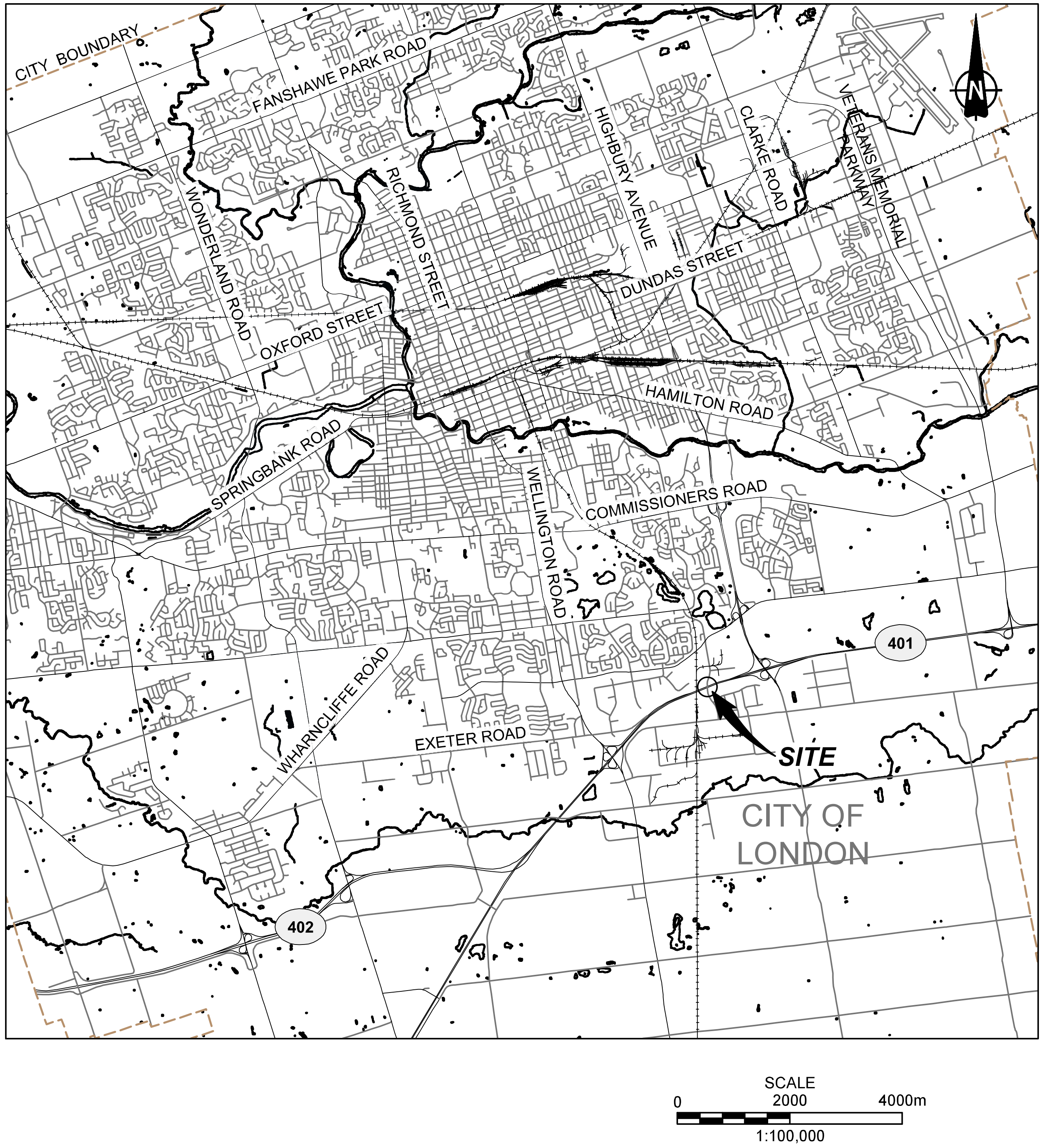
**SAMPLES**

| NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|
| 1 | SS | 14 | |
| 2 | SS | 5 | |
| 3 | SS | 6 | 0 12 58 30 |
| 4 | SS | 4 | |
| 5 | SS | 7 | |
| 6 | SS | 23 | |
| 7 | SS | 30 | |
| 8 | SS | 35 | 0 12 48 40 |
| 9 | SS | 30 | |
| 10 | SS | 29 | 1 11 55 33 |
| 11 | SS | 34 | |



UNIT WEIGHT γ kN/m³

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



## REFERENCE

PLAN BASED ON CITY OF LONDON CITY CD v.2011.

## NOTE

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

ALL LOCATIONS ARE APPROXIMATE.

PROJECT
POND MILLS ROAD OVERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3054-11-00

TITLE
## KEY PLAN

Golder Associates
LONDON, ONTARIO

| | | | |
|---|---|---|---|
| PROJECT No. | 12-1132-0076 | FILE No. | 1211320076-1001-F04001 |
| | | SCALE AS SHOWN | REV. 0 |
| CADD | AMG/LMK | June 7/13 | **FIGURE 1** |
| CHECK | | | |

Drawing file: 1211320076-1001-F04001.dwg Sep 17, 2013 - 8:57pm

MINISTRY OF TRANSPORTATION, ONTARIO

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

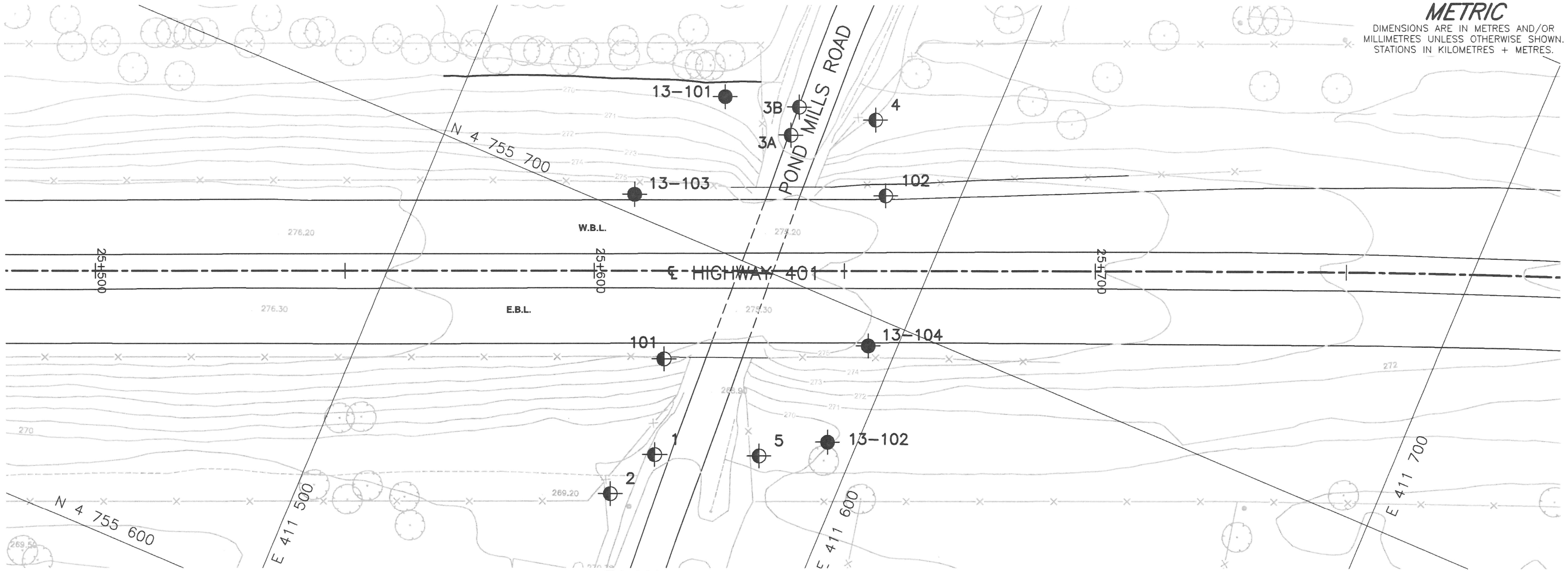
CONT No.
WP No. 3054-11-00

SHEET

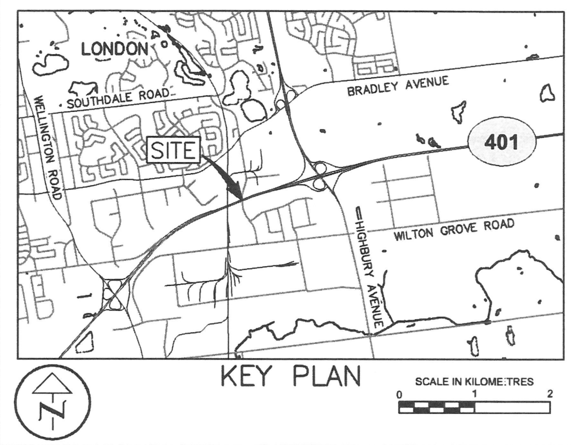
## POND MILLS ROAD OVERPASS
HIGHWAY 401 IMPROVEMENTS

BOREHOLE LOCATIONS AND SOIL STRATA

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



PLAN
SCALE
10 0 10m



KEY PLAN

## LEGEND

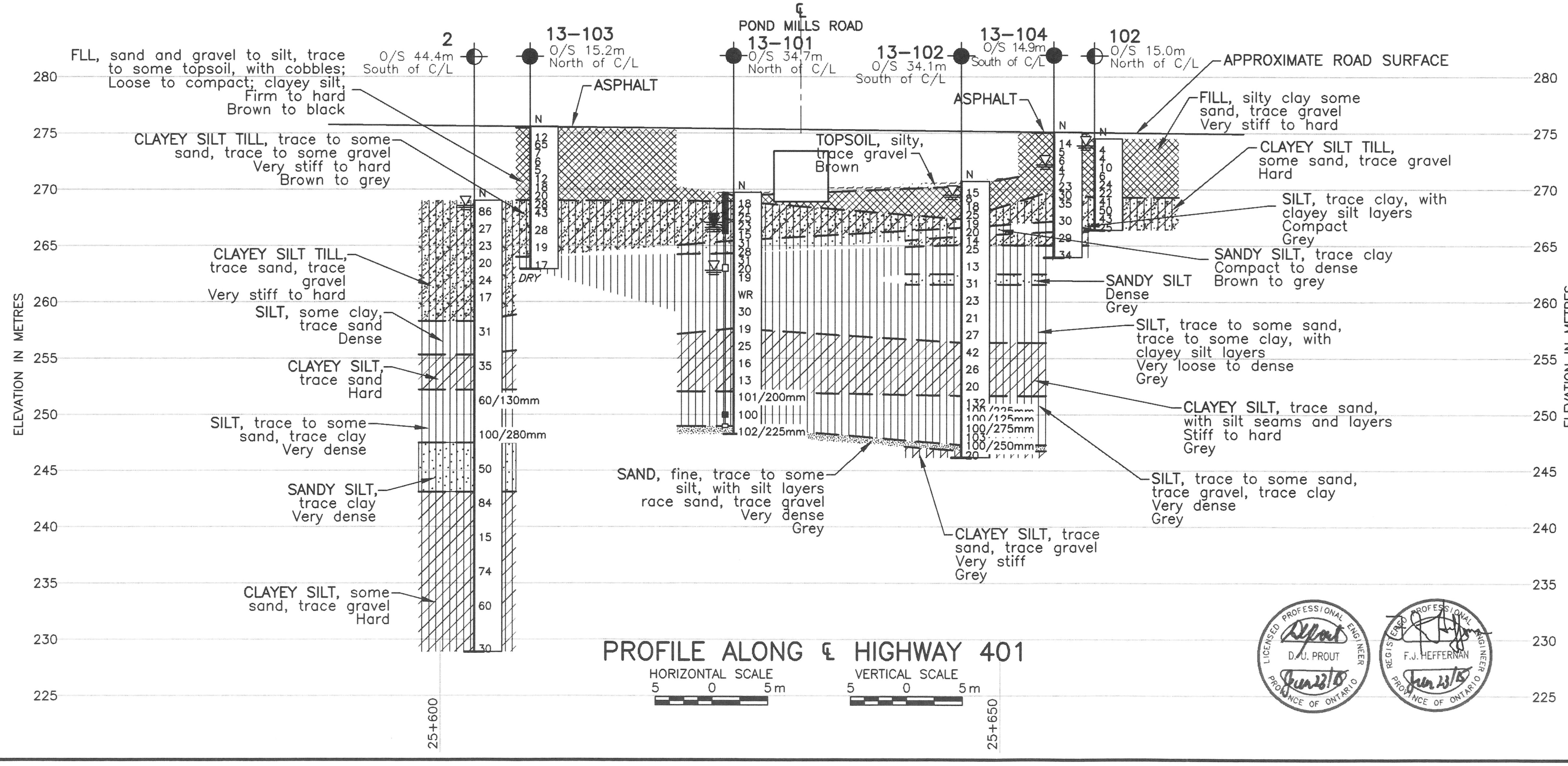
- ● Borehole – Current Investigation
- ⊕ Borehole (Geocres 40I14–111)
- ▮ Seal
- ▯ Standpipe
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ▼ WL measured on June 5, 2013
- ▽ WL encountered during drilling
- *DRY* Water level not established

| No. | ELEVATION | CO-ORDINATES (MTM ZONE 10) | |
|---|---|---|---|
| | | NORTHING | EASTING |
| 13–101 | 269.72 | 4 755 728.8 | 411 548.9 |
| 13–102 | 270.72 | 4 755 673.4 | 411 594.4 |
| 13–103 | 275.58 | 4 755 703.8 | 411 539.8 |
| 13–104 | 275.10 | 4 755 694.3 | 411 594.5 |
| Geocres 40I14–111 | | | |
| 1 | 268.3 | 4 755 657.6 | 411 563.5 |
| 2 | 269.0 | 4 755 647.0 | 411 558.4 |
| 3A | 269.0 | 4 755 726.9 | 411 564.0 |
| 3B | 269.0 | 4 755 732.7 | 411 563.4 |
| 4 | 269.5 | 4 755 736.3 | 411 578.5 |
| 5 | 269.5 | 4 755 665.5 | 411 582.8 |
| 101 | 274.7 | 4 755 675.9 | 411 557.8 |
| 102 | 274.5 | 4 755 723.2 | 411 586.2 |

## 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 based on City of London Digital Mapping Disc 2011 (converted to MTM ZONE 11



PROFILE ALONG ℄ HIGHWAY 401
HORIZONTAL SCALE
5 0 5m
VERTICAL SCALE
5 0 5m

D.J. PROUT
Jun 23/15

F.J. HEFFERNAN
Jun 23/15

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

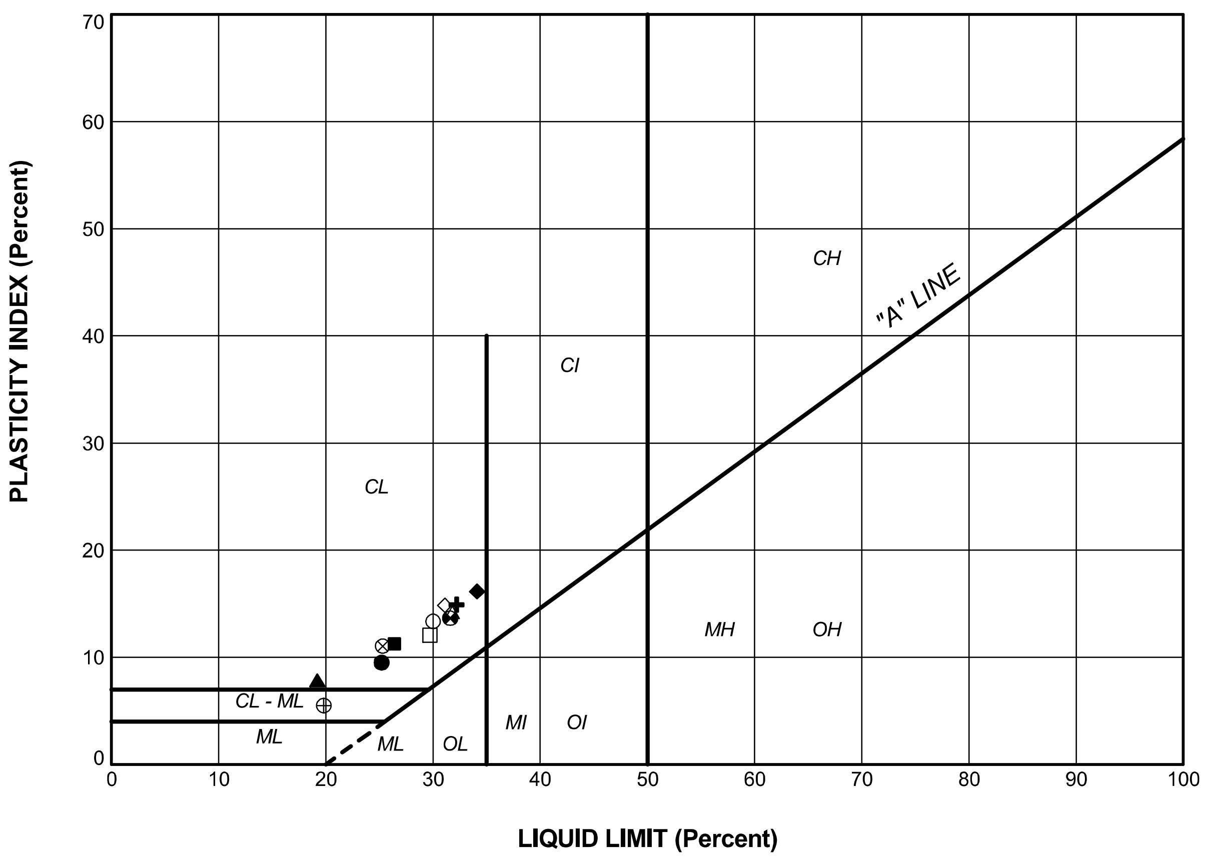
Geocres No. 40I14–157

| | | | |
|---|---|---|---|
| HWY. 401 | | PROJECT NO. 12–1132–0076 | DIST. |
| SUBM'D. NG | CHKD. NAG | DATE: June 12/13 | SITE: 19–372 |
| DRAWN: WDF/LMK | CHKD. DUP | APPD. FJH | DWG. 1 |

PLOT DATE: June 02, 2015
FILENAME: N:\active\2012\1132 - Geo\1132-0000\12-1132-0076 DILLON-11 STRUCTURES-3011-E-0046\Drafting\AutoCAD files\1211320076-1001-004001.dwg

# APPENDIX A

## Laboratory Test Data



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

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

## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | LL(%) | PL(%) | PI |
|---|---|---|---|---|---|
| FILL, clayey silt | | | | | |
| ● | 13-103 | 3 | 25.2 | 15.7 | 9.5 |
| ■ | 13-103 | 6 | 26.4 | 15.2 | 11.3 |
| ▲ | 13-104 | 3 | 19.2 | 11.4 | 7.9 |
| CLAYEY SILT TILL | | | | | |
| ✚ | 13-101 | 4 | 32.2 | 17.3 | 14.9 |
| ◆ | 13-102 | 5 | 34.1 | 18.0 | 16.2 |
| ◇ | 13-103 | 10 | 31.1 | 16.3 | 14.9 |
| ○ | 13-103 | 12 | 30.0 | 16.7 | 13.4 |
| △ | 13-104 | 8 | 31.8 | 17.6 | 14.2 |
| ⊗ | 13-104 | 10 | 25.3 | 14.3 | 11.1 |
| CLAYEY SILT | | | | | |
| ⊕ | 13-101 | 14 | 19.8 | 14.3 | 5.5 |
| □ | 13-102 | 15 | 29.7 | 17.7 | 12.1 |
| ⨂ | 13-102 | 16 | 31.6 | 18.0 | 13.7 |

LDN_MTO_PI GLDR_LDN.GDT

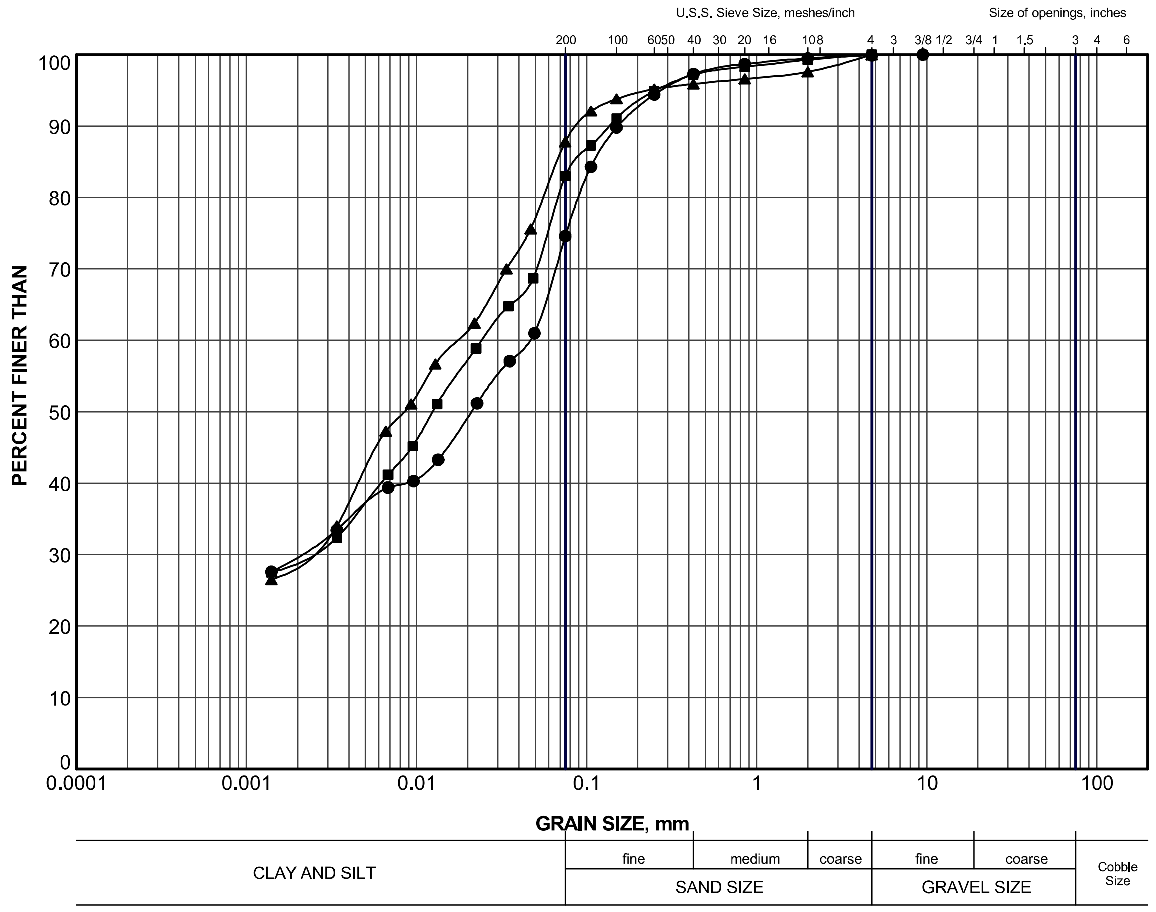
PROJECT
POND MILLS ROAD OVERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3054-11-00

TITLE
## PLASTICITY CHART

Golder Associates
LONDON, ONTARIO

| | | | |
|---|---|---|---|
| PROJECT No.12-1132-0076-1001 | | | FILE No. 1211320076-1001-F040A1 |
| | | | SCALE N/A REV. |
| DRAWN | LMK | Jun 12/13 | FIGURE A-1 |
| CHECK | | | |

---



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|---|---|---|---|
| ● | 13-103 | 3 | 273.2 |
| ■ | 13-103 | 6 | 270.9 |
| ▲ | 13-104 | 3 | 272.6 |

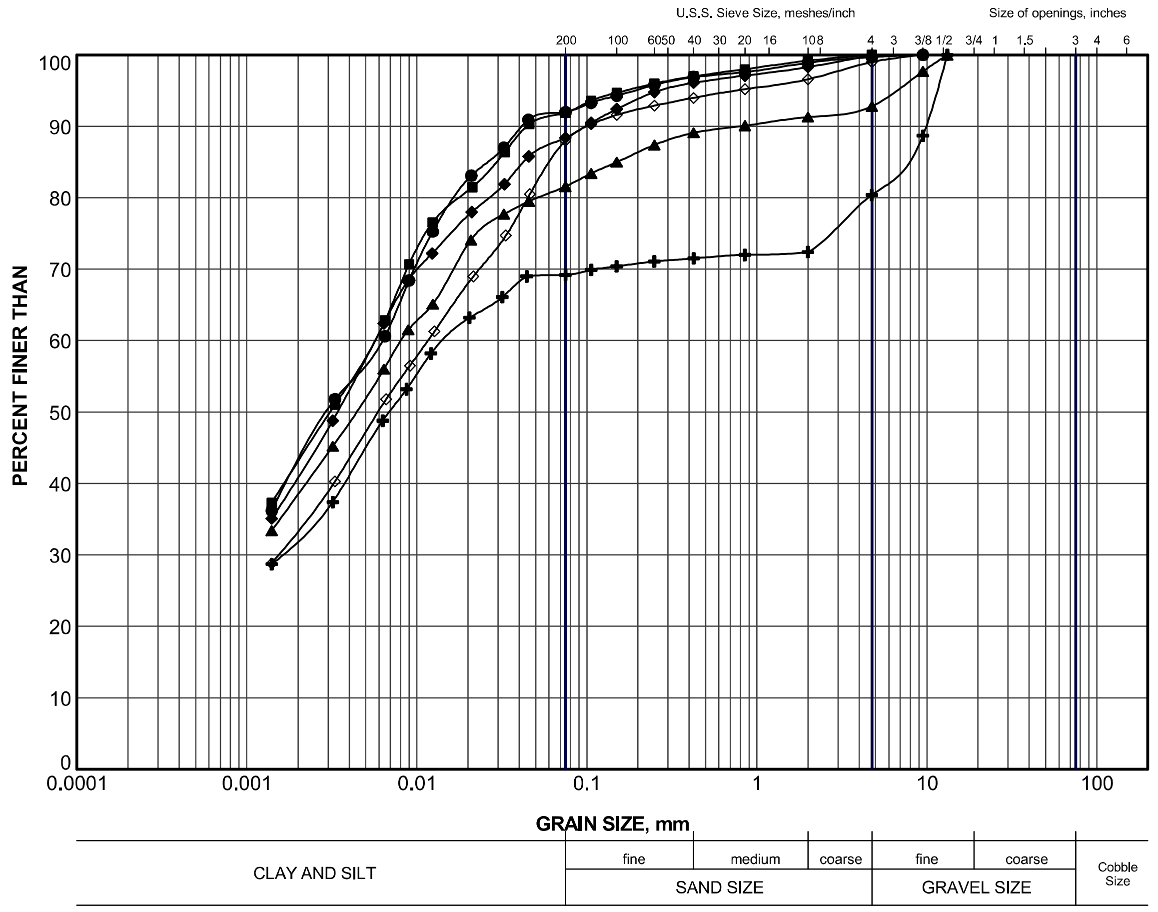
LDN_MTO_GSD GLDR_LDN.GDT

PROJECT
POND MILLS ROAD OVERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3054-11-00

TITLE
## GRAIN SIZE DISTRIBUTION
### CLAYEY SILT FILL

Golder Associates
LONDON, ONTARIO

| | | | |
|---|---|---|---|
| PROJECT No.12-1132-0076-1001 | | | FILE No. 1211320076-1001-F040A2 |
| | | | SCALE N/A REV. |
| DRAWN | LMK | Jun 12/13 | FIGURE A-2 |
| CHECK | | | |



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|---|---|---|---|
| ● | 13-101 | 4 | 266.8 |
| ■ | 13-102 | 5 | 267.0 |
| ▲ | 13-103 | 10 | 267.9 |
| ✚ | 13-103 | 12 | 264.8 |
| ◆ | 13-104 | 8 | 268.8 |
| ◇ | 13-104 | 10 | 265.7 |

PROJECT
POND MILLS ROAD OVERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3054-11-00

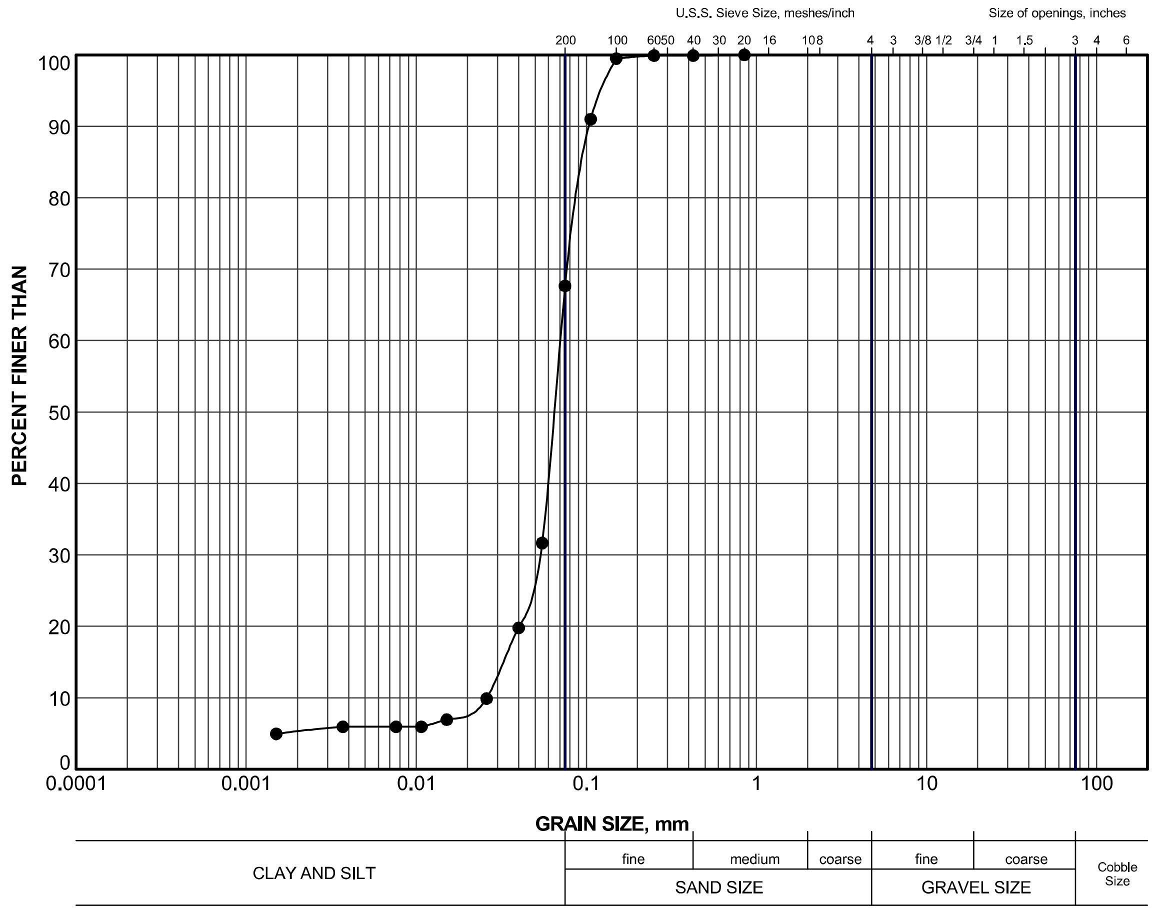
TITLE
# GRAIN SIZE DISTRIBUTION
## CLAYEY SILT TILL

Golder Associates
LONDON, ONTARIO

| PROJECT No.12-1132-0076-1001 | | | FILE No. 1211320076-1001-F040A3 | |
|---|---|---|---|---|
| | | | SCALE N/A | REV. |
| DRAWN | LMK | Jun 12/13 | **FIGURE A-3** | |
| CHECK | | | | |

LDN_MTO_GSD_GLDR_LDN.GDT

---



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|---|---|---|---|
| ● | 13-102 | 10 | 261.7 |

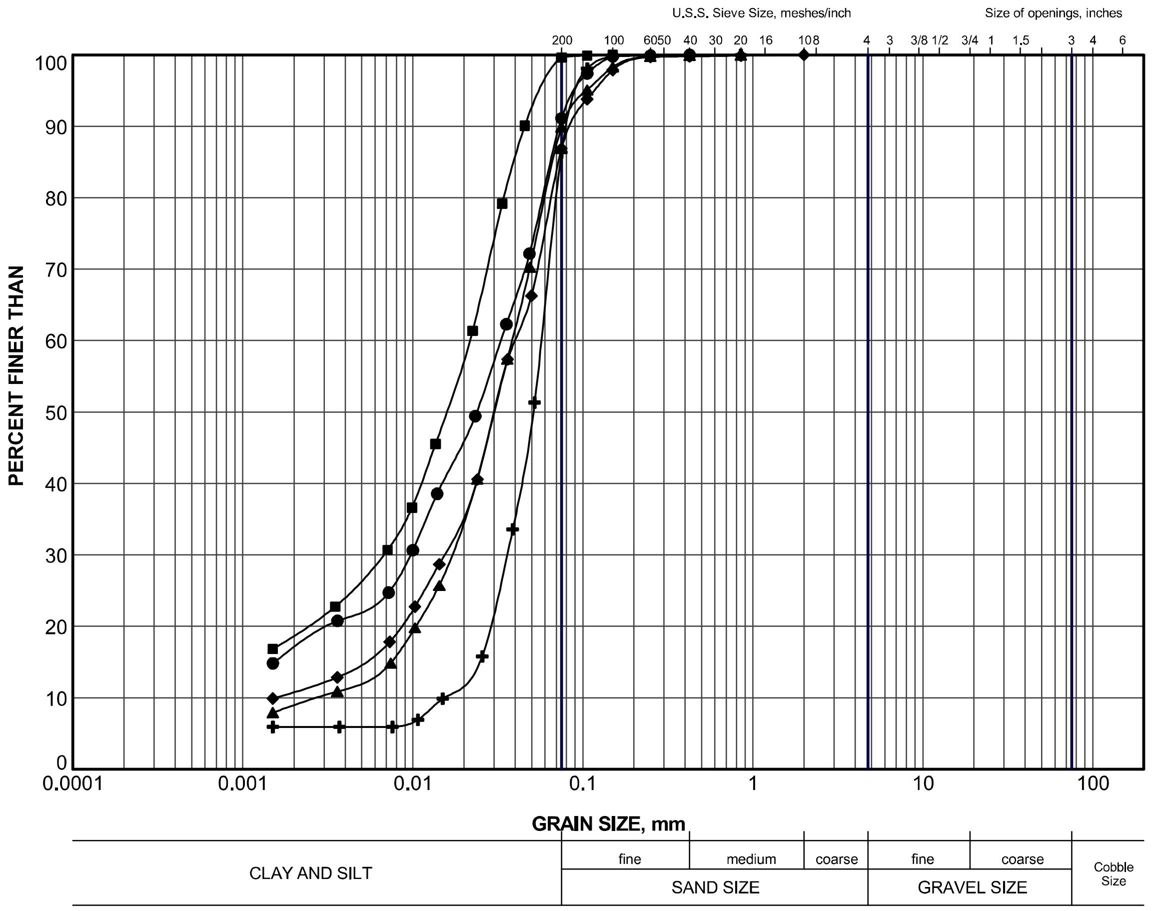
PROJECT
POND MILLS ROAD OVERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3054-11-00

TITLE
# GRAIN SIZE DISTRIBUTION
## SANDY SILT

Golder Associates
LONDON, ONTARIO

| PROJECT No.12-1132-0076-1001 | | | FILE No. 1211320076-1001-F040A4 | |
|---|---|---|---|---|
| | | | SCALE N/A | REV. |
| DRAWN | LMK | Jun 12/13 | **FIGURE A-4** | |
| CHECK | | | | |

LDN_MTO_GSD_GLDR_LDN.GDT



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|---|---|---|---|
| ● | 13-101 | 10 | 262.2 |
| ■ | 13-101 | 12 | 259.1 |
| ▲ | 13-101 | 17 | 251.5 |
| ✚ | 13-102 | 12 | 258.6 |
| ◆ | 13-102 | 19 | 249.6 |

PROJECT
POND MILLS ROAD OVERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3054-11-00

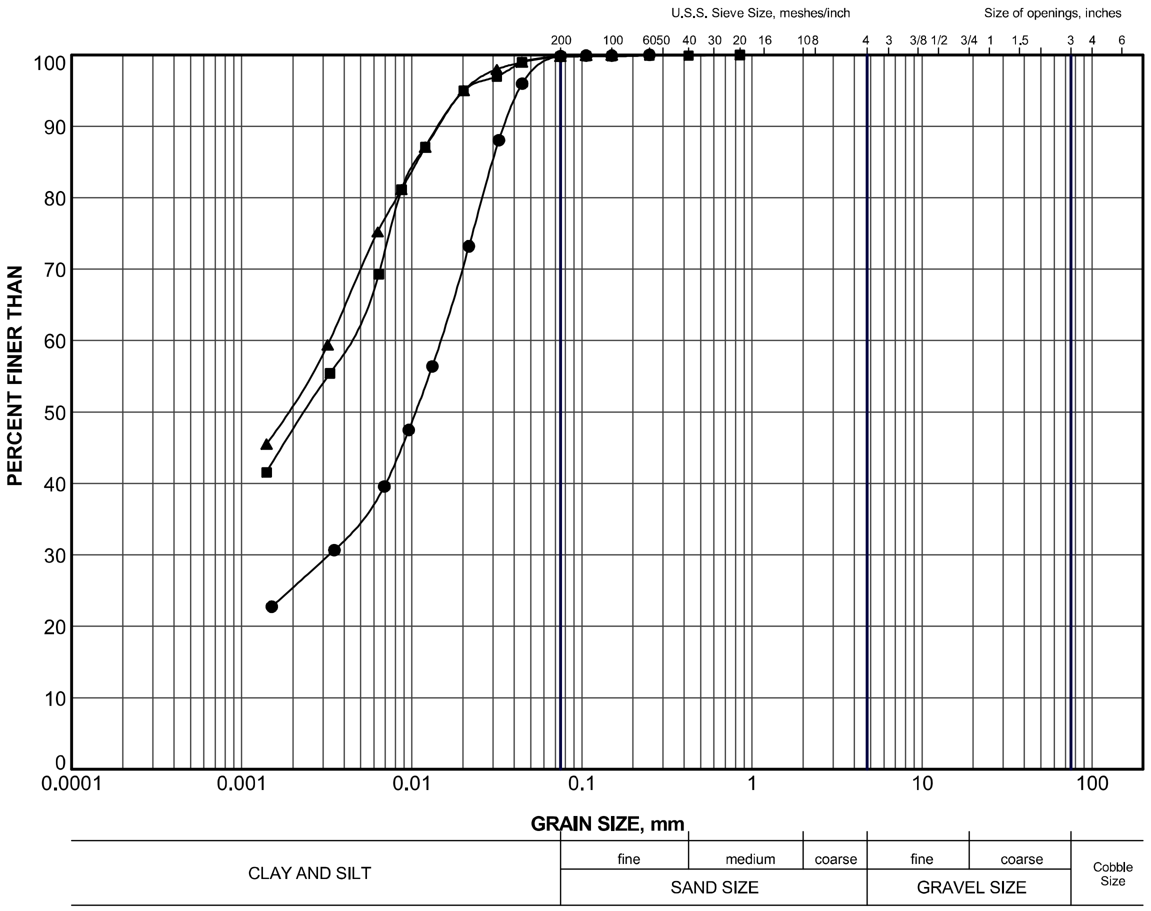
TITLE
## GRAIN SIZE DISTRIBUTION
## SILT

Golder Associates
LONDON, ONTARIO

| | | | | |
|---|---|---|---|---|
| PROJECT No.12-1132-0076-1001 | | | FILE No. 1211320076-1001-F040A5 | |
| | | | SCALE N/A | REV. |
| DRAWN | LMK | Jun 07/13 | **FIGURE A-5** | |
| CHECK | | | | |

LDN_MTO_GSD GLDR_LDN.GDT

---



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|---|---|---|---|
| ● | 13-101 | 14 | 256.1 |
| ■ | 13-102 | 15 | 253.9 |
| ▲ | 13-102 | 16 | 252.5 |

PROJECT
POND MILLS ROAD OVERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3054-11-00

TITLE
## GRAIN SIZE DISTRIBUTION
## CLAYEY SILT

Golder Associates
LONDON, ONTARIO

| | | | | |
|---|---|---|---|---|
| PROJECT No.12-1132-0076-1001 | | | FILE No. 1211320076-1001-F040A6 | |
| | | | SCALE N/A | REV. |
| DRAWN | LMK | Jun 12/13 | **FIGURE A-6** | |
| CHECK | | | | |

LDN_MTO_GSD GLDR_LDN.GDT

# APPENDIX C

## C.1 SYMBOLS AND TERMS USED ON BOREHOLE RECORDS

## C.2 BOREHOLE RECORDS

# SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

## SOIL DESCRIPTION

**Terminology describing common soil genesis:**

| | |
|---|---|
| *Rootmat* | - vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface |
| *Topsoil* | - mixture of soil and humus capable of supporting vegetative growth |
| *Peat* | - mixture of visible and invisible fragments of decayed organic matter |
| *Till* | - unstratified glacial deposit which may range from clay to boulders |
| *Fill* | - material below the surface identified as placed by humans (excluding buried services) |

**Terminology describing soil structure:**

| | |
|---|---|
| *Desiccated* | - having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc. |
| *Fissured* | - having cracks, and hence a blocky structure |
| *Varved* | - composed of regular alternating layers of silt and clay |
| *Stratified* | - composed of alternating successions of different soil types, e.g. silt and sand |
| *Layer* | - > 75 mm in thickness |
| *Seam* | - 2 mm to 75 mm in thickness |
| *Parting* | - < 2 mm in thickness |

**Terminology describing soil types**:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

**Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):**
Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

| | |
|---|---|
| *Trace, or occasional* | Less than 10% |
| *Some* | 10-20% |
| *Frequent* | > 20% |

**Terminology describing compactness of cohesionless soils:**
The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

| **Compactness Condition** | **SPT N-Value** |
|---|---|
| *Very Loose* | <4 |
| *Loose* | 4-10 |
| *Compact* | 10-30 |
| *Dense* | 30-50 |
| *Very Dense* | >50 |

**Terminology describing consistency of cohesive soils:**
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

| **Consistency** | **Undrained Shear Strength** | | **Approximate SPT N-Value** |
|---|---|---|---|
| | **kips/sq.ft.** | **kPa** | |
| *Very Soft* | <0.25 | <12.5 | <2 |
| *Soft* | 0.25 - 0.5 | 12.5 - 25 | 2-4 |
| *Firm* | 0.5 - 1.0 | 25 - 50 | 4-8 |
| *Stiff* | 1.0 - 2.0 | 50 – 100 | 8-15 |
| *Very Stiff* | 2.0 - 4.0 | 100 - 200 | 15-30 |
| *Hard* | >4.0 | >200 | >30 |

## ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

**Terminology describing rock quality:**

| RQD | Rock Mass Quality |
|---|---|
| 0-25 | *Very Poor Quality* |
| 25-50 | *Poor Quality* |
| 50-75 | *Fair Quality* |
| 75-90 | *Good Quality* |
| 90-100 | *Excellent Quality* |

| Alternate (Colloquial) Rock Mass Quality | |
|---|---|
| *Very Severely Fractured* | *Crushed* |
| *Severely Fractured* | *Shattered or Very Blocky* |
| *Fractured* | *Blocky* |
| *Moderately Jointed* | *Sound* |
| *Intact* | *Very Sound* |

**RQD (Rock Quality Designation)** denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

**SCR (Solid Core Recovery)** denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

**Fracture Index (FI)** is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

**Terminology describing rock with respect to discontinuity and bedding spacing:**

| Spacing (mm) | Discontinuities | Bedding |
|---|---|---|
| >6000 | *Extremely Wide* | - |
| 2000-6000 | *Very Wide* | *Very Thick* |
| 600-2000 | *Wide* | *Thick* |
| 200-600 | *Moderate* | *Medium* |
| 60-200 | *Close* | *Thin* |
| 20-60 | *Very Close* | *Very Thin* |
| <20 | *Extremely Close* | *Laminated* |
| <6 | - | *Thinly Laminated* |

**Terminology describing rock strength:**

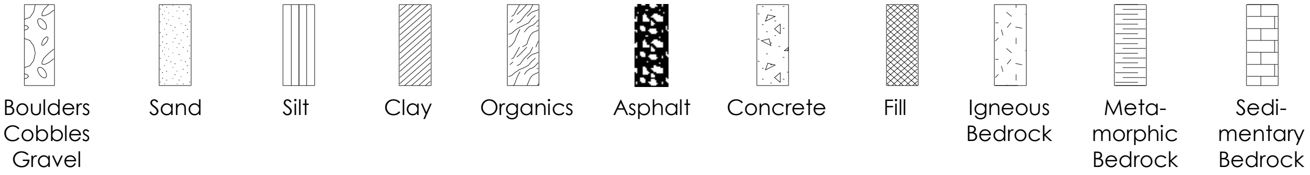
| Strength Classification | Grade | Unconfined Compressive Strength (MPa) |
|---|---|---|
| *Extremely Weak* | R0 | <1 |
| *Very Weak* | R1 | 1 – 5 |
| *Weak* | R2 | 5 – 25 |
| *Medium Strong* | R3 | 25 – 50 |
| *Strong* | R4 | 50 – 100 |
| *Very Strong* | R5 | 100 – 250 |
| *Extremely Strong* | R6 | >250 |

**Terminology describing rock weathering:**

| Term | Symbol | Description |
|---|---|---|
| *Fresh* | W1 | No visible signs of rock weathering. Slight discoloration along major discontinuities |
| *Slightly* | W2 | Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored. |
| *Moderately* | W3 | Less than half the rock is decomposed and/or disintegrated into soil. |
| *Highly* | W4 | More than half the rock is decomposed and/or disintegrated into soil. |
| *Completely* | W5 | All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact. |
| *Residual Soil* | W6 | All the rock converted to soil. Structure and fabric destroyed. |

## STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



## SAMPLE TYPE

| | |
|---|---|
| *SS* | Split spoon sample (obtained by performing the Standard Penetration Test) |
| *ST* | Shelby tube or thin wall tube |
| *DP* | Direct-Push sample (small diameter tube sampler hydraulically advanced) |
| *PS* | Piston sample |
| *BS* | Bulk sample |
| *HQ, NQ, BQ, etc.* | Rock core samples obtained with the use of standard size diamond coring bits. |

## WATER LEVEL MEASUREMENT

| | |
|---|---|
| ▼ | measured in standpipe, piezometer, or well |
| ▽ | inferred |

## RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

## N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

## DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to ‘A’ size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

## OTHER TESTS

| | |
|---|---|
| *S* | Sieve analysis |
| *H* | Hydrometer analysis |
| *k* | Laboratory permeability |
| *γ* | Unit weight |
| *$G_s$* | Specific gravity of soil particles |
| *CD* | Consolidated drained triaxial |
| *CU* | Consolidated undrained triaxial with pore pressure measurements |
| *UU* | Unconsolidated undrained triaxial |
| *DS* | Direct Shear |
| *C* | Consolidation |
| *$Q_U$* | Unconfined compression |
| *$I_p$* | Point Load Index ($I_p$ on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm) |

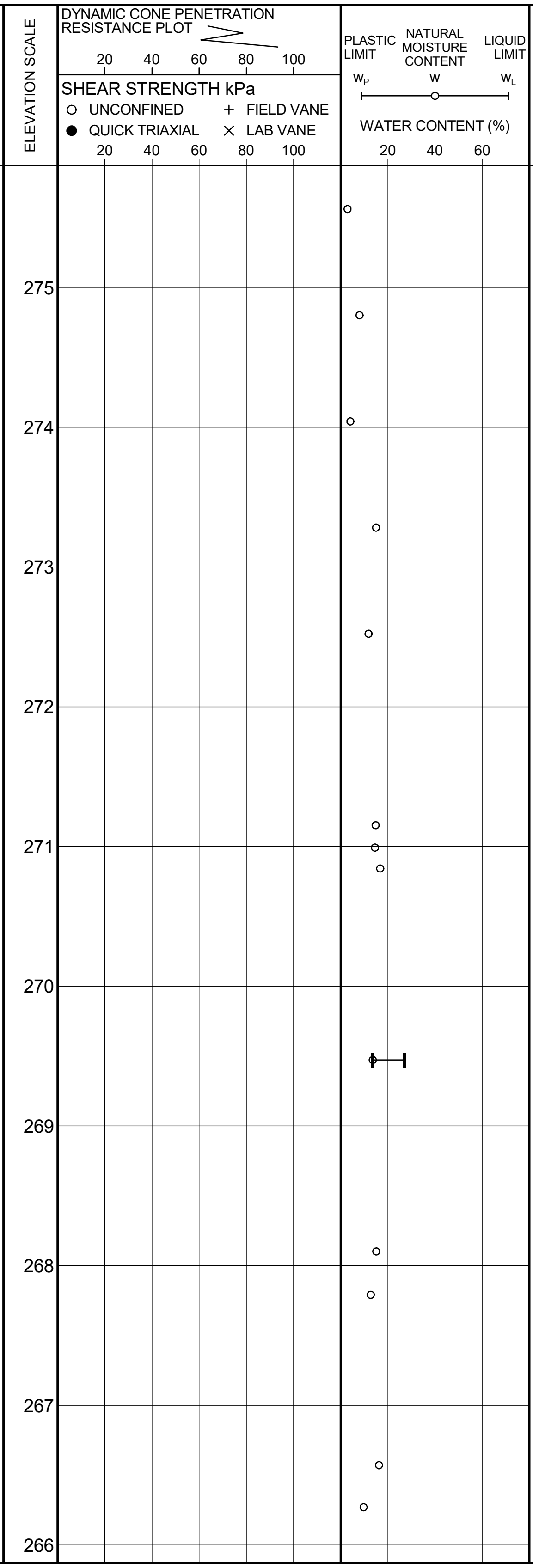
| | |
|---|---|
| ↧ | Single packer permeability test; test interval from depth shown to bottom of borehole |
| ⌶ | Double packer permeability test; test interval as indicated |
| ⫯ | Falling head permeability test using casing |
| ▿ | Falling head permeability test using well point or piezometer |

# RECORD OF BOREHOLE No PM-01

1 OF 2 **METRIC**

W.P. 3032-11-00
LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario
ORIGINATED BY WT
DIST West HWY 401
BOREHOLE TYPE Solid Stem Augers
COMPILED BY JM
DATUM Geodetic
DATE 2022.07.13 - 2022.07.14 LATITUDE 42.9334885 LONGITUDE -81.1924381
CHECKED BY GR

| ELEV<br>DEPTH | SOIL PROFILE DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| 275.9 | | | | | | |
| 0.0 | 280 mm ASPHALT | 1 | SS | 40 | | |
| 275.6<br>0.3 | FILL: Silty SAND with Gravel (SM). Contains clayey inclusions.<br>Brown<br>Compact to dense<br>Dry | | | | | |
| | | 2 | SS | 15 | | |
| | | 3 | SS | 16 | | 20 59 13 8 |
| 273.6<br>2.3 | FILL: CLAYEY SILT with Sand (CL), trace gravel<br>Brown<br>Stiff to hard<br>Moist | 4 | SS | 15 | | PP=2.25<br>Su= 121 kPa |
| | | 5 | SS | 25 | | |
| | 125 mm seam of fine sand in SS6 | 6 | SS | 18 | | PP=0.75<br>Su= 40 kPa |
| | Layer of black organic material in SS7 | 7 | SS | 29 | | 2 36 33 29<br>PP>4.5<br>Su> 241 kPa |
| | | 8 | SS | 38 | | PP>4.5<br>Su> 241 kPa |
| 267.2<br>8.7 | CLAYEY SILT to SILTY CLAY with Sand (CL), trace gravel<br>Brown to grey<br>Very stiff to hard<br>Moist | | | | | |
| | | 9 | SS | 31 | | PP=4.25<br>Su= 228 kPa |



Continued Next Page

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

# RECORD OF BOREHOLE No PM-01

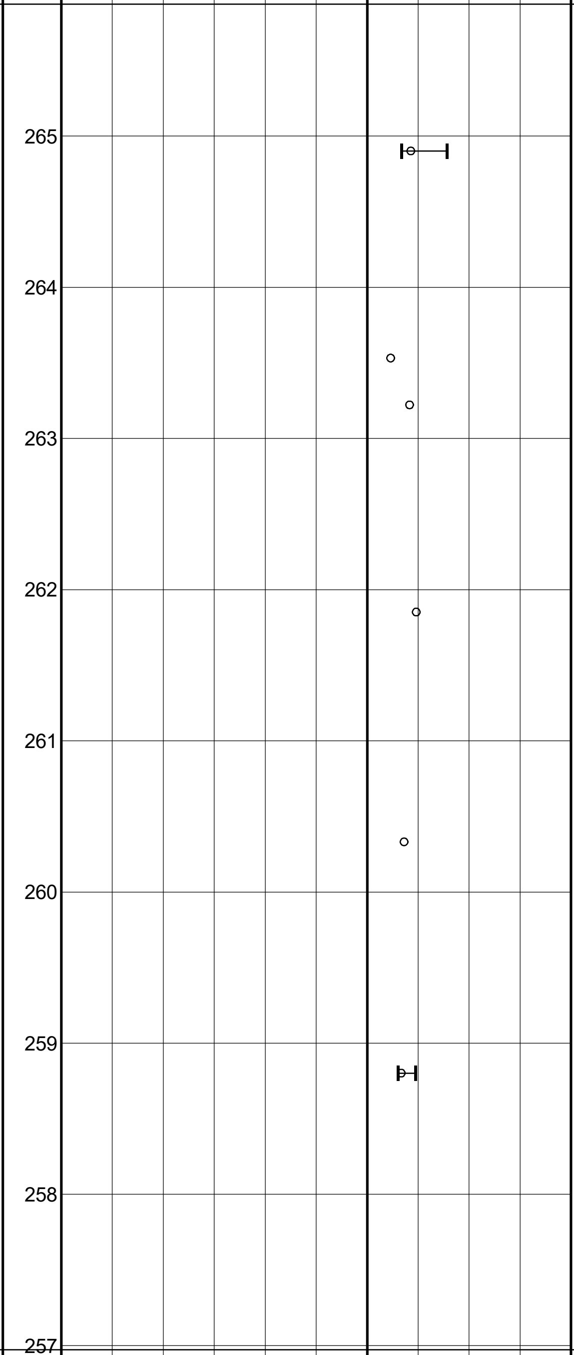
2 OF 2

**METRIC**

W.P. 3032-11-00
LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario
ORIGINATED BY WT
DIST West
HWY 401
BOREHOLE TYPE Solid Stem Augers
COMPILED BY JM
DATUM Geodetic
DATE 2022.07.13 - 2022.07.14
LATITUDE 42.9334885
LONGITUDE -81.1924381
CHECKED BY GR

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 (%) 20 40 60
UNIT WEIGHT γ kN/m³

| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | | |
| | CLAYEY SILT to SILTY CLAY with Sand (CL), trace gravel<br>Brown to grey<br>Very stiff to hard<br>Moist *(continued)*<br>Grey below 10.7m | | | | ∇ | |
| | | 10 | SS | 18 | | 2 13 35 50<br>PP=2.25<br>Su= 121 kPa |
| | | 11 | SS | 20 | | PP=1.75<br>Su= 94 kPa |
| | | 12 | SS | 13 | | PP=1.0<br>Su= 54 kPa |
| 261.1<br>14.8 | CLAYEY SILT (CL-ML), trace sand<br>Grey<br>Hard<br>Moist | | | | | |
| | | 13 | SS | 43 | | PP=3.25<br>Su= 174 kPa |
| | | 14 | SS | 61 | | 0 2 64 34<br>PP>4.5<br>Su> 241 kPa |
| | | 15 | SS | 51 | | PP=4.25<br>Su= 228 kPa |
| 257.0<br>18.9 | END OF BOREHOLE<br>Groundwater level and cave-in measured at approximately 18.0 m below grade, respectively. | | | | | |



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

ONTARIO MTO 165001239_MTO_HWY_401_HIGHBURY.GPJ ONTARIO MTO.GDT 1/20/23

# RECORD OF BOREHOLE No PM-02

1 OF 5

**METRIC**

| | | | |
|---|---|---|---|
| W.P. 3032-11-00 | LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario | | ORIGINATED BY JM/AS |
| DIST West HWY 401 | BOREHOLE TYPE Hollow Stem Augers | | COMPILED BY RR |
| DATUM Geodetic | DATE 2022.07.15 - 2022.07.18 | LATITUDE 42.9335766 LONGITUDE -81.1921493 | CHECKED BY GR |

## SOIL PROFILE

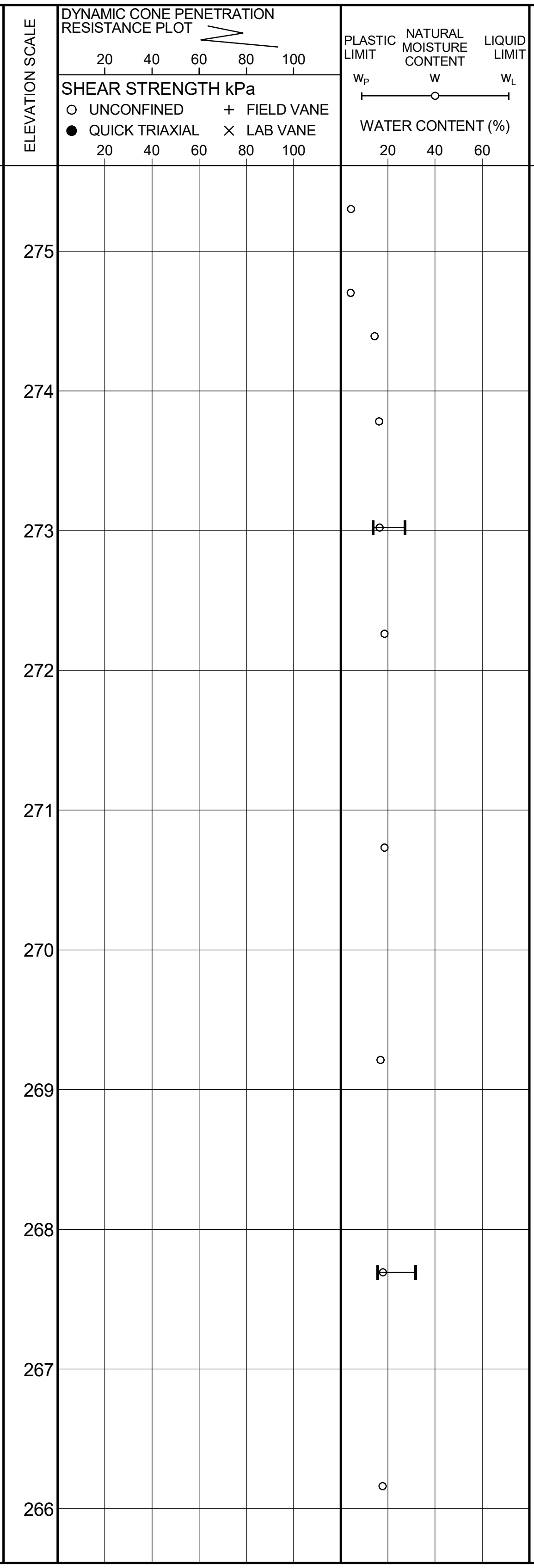
| ELEV<br>DEPTH | DESCRIPTION |
|---|---|
| 275.6 | |
| 0.0 | 280 mm ASPHALT |
| 275.3<br>0.3 | FILL: SILTY SAND (SM), trace gravel. Contains construction debris.<br>Brown<br>Compact<br>Dry |
| 274.5<br>1.1 | FILL: Sandy CLAYEY SILT (CL), trace gravel and organics<br>Brown<br>Stiff to very stiff<br>Dry to moist |
| 268.0<br>7.6 | CLAYEY SILT (CL-ML) to SILTY CLAY (CL), some sand, trace gravel<br>Brown<br>Very stiff to hard<br>Moist |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|
| 1 | SS | 26 | |
| 2 | SS | 14 | PP=2.25<br>Su= 121 kPa |
| 3 | SS | 12 | PP=0.75<br>Su= 40 kPa |
| 4 | SS | 7 | 4 30 30 35<br>PP=0.75<br>Su= 40 kPa<br>Su > 100 kPa |
| 5 | SS | 14 | PP=2.5<br>Su= 134 kPa |
| 6 | SS | 15 | PP=2.25<br>Su= 121 kPa |
| 7 | SS | 23 | PP=3.75<br>Su= 201 kPa |
| 8 | SS | 19 | 1 17 34 49<br>PP=3.25<br>Su= 174 kPa |
| 9 | SS | 28 | PP=3.75<br>Su= 174 kPa |

GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m³



Continued Next Page

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

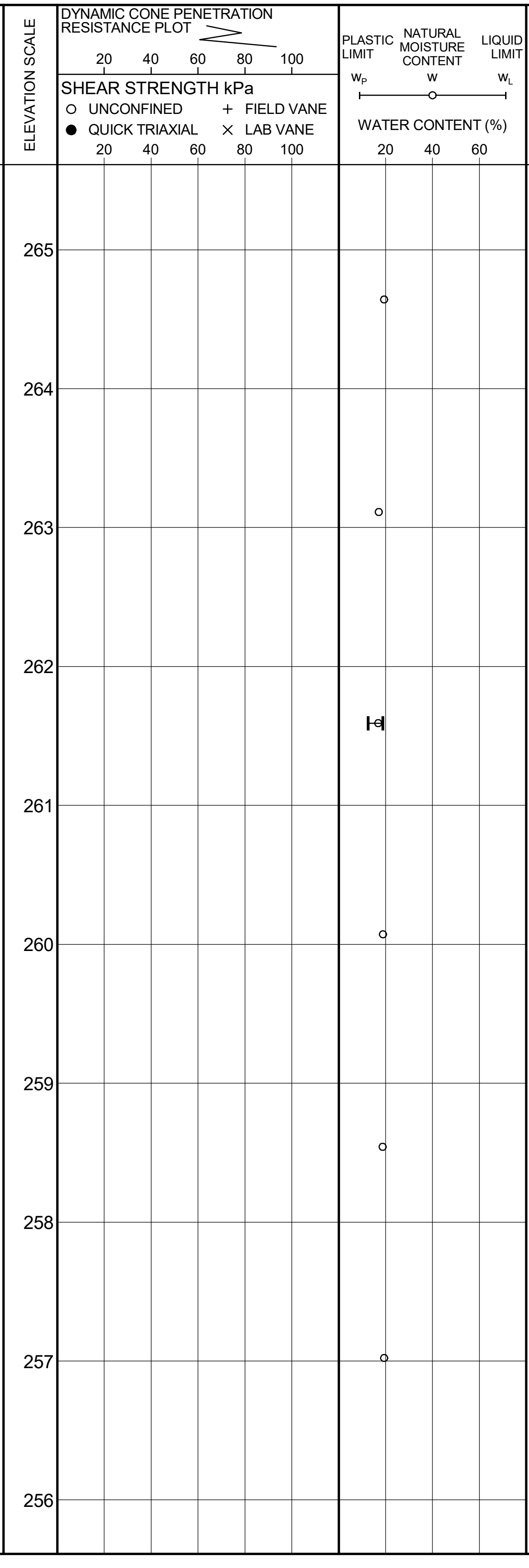
# RECORD OF BOREHOLE No PM-02

2 OF 5

**METRIC**

W.P. 3032-11-00
LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario
ORIGINATED BY JM/AS
DIST West HWY 401
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY RR
DATUM Geodetic
DATE 2022.07.15 - 2022.07.18
LATITUDE 42.9335766
LONGITUDE -81.1921493
CHECKED BY GR

| SOIL PROFILE<br>ELEV<br>DEPTH | DESCRIPTION | SAMPLES<br>NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
|---|---|---|---|---|---|---|
| | CLAYEY SILT (CL-ML) to SILTY CLAY (CL), some sand, trace gravel<br>Brown<br>Very stiff to hard<br>Moist *(continued)* | | | | | |
| | | 10 | SS | 16 | | PP=2.25<br>Su= 121 kPa |
| | | | | | ∇ | |
| | Grey below 12.2m | 11 | SS | 34 | | PP=2.5<br>Su= 134 kPa |
| | | 12 | SS | 27 | | 0 1 66 33 |
| 260.7<br>14.9 | SILT to SILT with Sand (ML), some clay<br>Grey<br>Moist to wet<br>Compact to very dense | | | | | |
| | | 13 | SS | 44 | | |
| | | 14 | SS | 43 | | |
| | | 15 | SS | 45 | | |



UNIT WEIGHT γ kN/m³

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

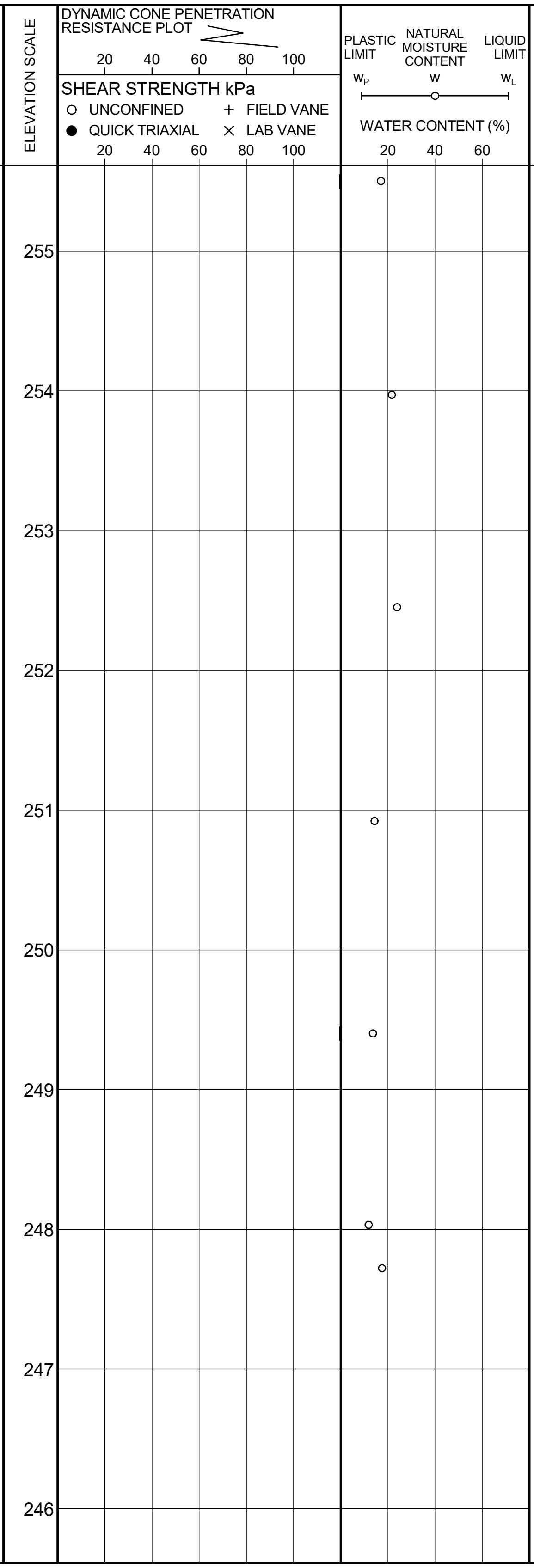
# RECORD OF BOREHOLE No PM-02

3 OF 5

**METRIC**

W.P. 3032-11-00
LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario
ORIGINATED BY JM/AS
DIST West HWY 401
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY RR
DATUM Geodetic
DATE 2022.07.15 - 2022.07.18
LATITUDE 42.9335766
LONGITUDE -81.1921493
CHECKED BY GR

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | SAMPLES NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|
| | SILT to SILT with Sand (ML), some clay<br>Grey<br>Moist to wet<br>Compact to very dense *(continued)* | 16 | SS | 58 | | | 0 | 1 | 76 | 23 |
| | | | | | | | Non-Plastic | | | |
| | | 17 | SS | 20 | | | | | | |
| | | 18 | SS | 20 | | | | | | |
| | | 19 | SS | 153 | | | | | | |
| | | 20 | SS | 102 | | | 0 | 20 | 60 | 20 |
| | | | | | | | Non-Plastic | | | |
| | | 21 | SS | 157 | | | | | | |
| 246.0<br>29.6 | | | | | | | | | | |



Continued Next Page

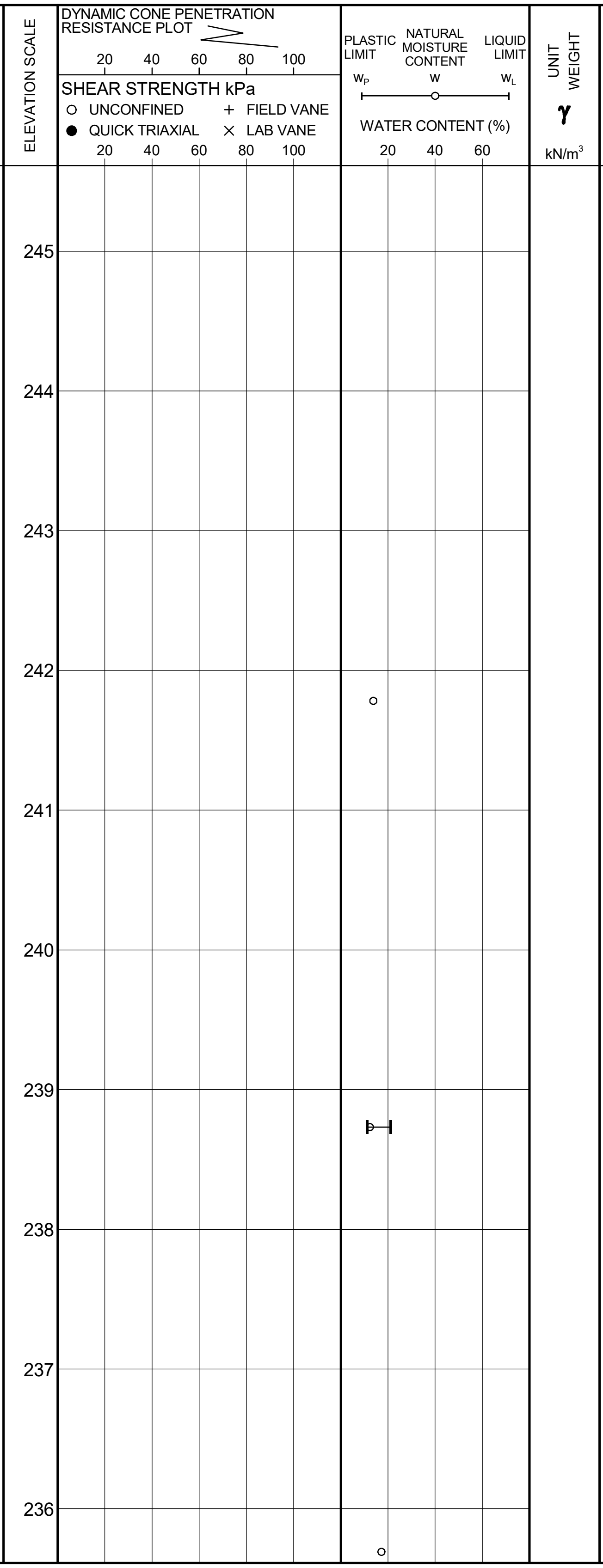
+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No PM-02

4 OF 5 **METRIC**

W.P. 3032-11-00
LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario
ORIGINATED BY JM/AS
DIST West HWY 401
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY RR
DATUM Geodetic
DATE 2022.07.15 - 2022.07.18 LATITUDE 42.9335766 LONGITUDE -81.1921493
CHECKED BY GR

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| | CLAYEY SILT with Sand (CL-ML to CL), trace gravel (TILL)<br>Grey<br>Hard<br>Moist *(continued)* | | | | | |
| | | 22 | SS | 32 | | PP=2.25<br>Su= 121 kPa |
| | | 23 | SS | 38 | | PP=2.25<br>Su= 121 kPa |
| | | 24 | SS | 51 | | 2 21 41 37<br>PP=2.25<br>Su= 121 kPa |
| | | 25 | SS | 38 | | |



Continued Next Page

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

# RECORD OF BOREHOLE No PM-02

5 OF 5 **METRIC**

W.P. 3032-11-00 LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario ORIGINATED BY JM/AS
DIST West HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY RR
DATUM Geodetic DATE 2022.07.15 - 2022.07.18 LATITUDE 42.9335766 LONGITUDE -81.1921493 CHECKED BY GR

| SOIL PROFILE | | | SAMPLES | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV<br>DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100<br>SHEAR STRENGTH kPa<br>○ UNCONFINED + FIELD VANE<br>● QUICK TRIAXIAL × LAB VANE<br>20 40 60 80 100 | PLASTIC LIMIT $w_P$ NATURAL MOISTURE CONTENT $w$ LIQUID LIMIT $w_L$<br>WATER CONTENT (%)<br>20 40 60 | UNIT WEIGHT $\gamma$ kN/m$^3$ | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
| 235.4 | | | | | | | | | | | PP=1.75<br>Su= 94 kPa |
| 40.2 | END OF BOREHOLE | | | | | | | | | | |

ONTARIO MTO 165001239_MTO_HWY_401_HIGHBURY.GPJ ONTARIO MTO.GDT 1/20/23

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

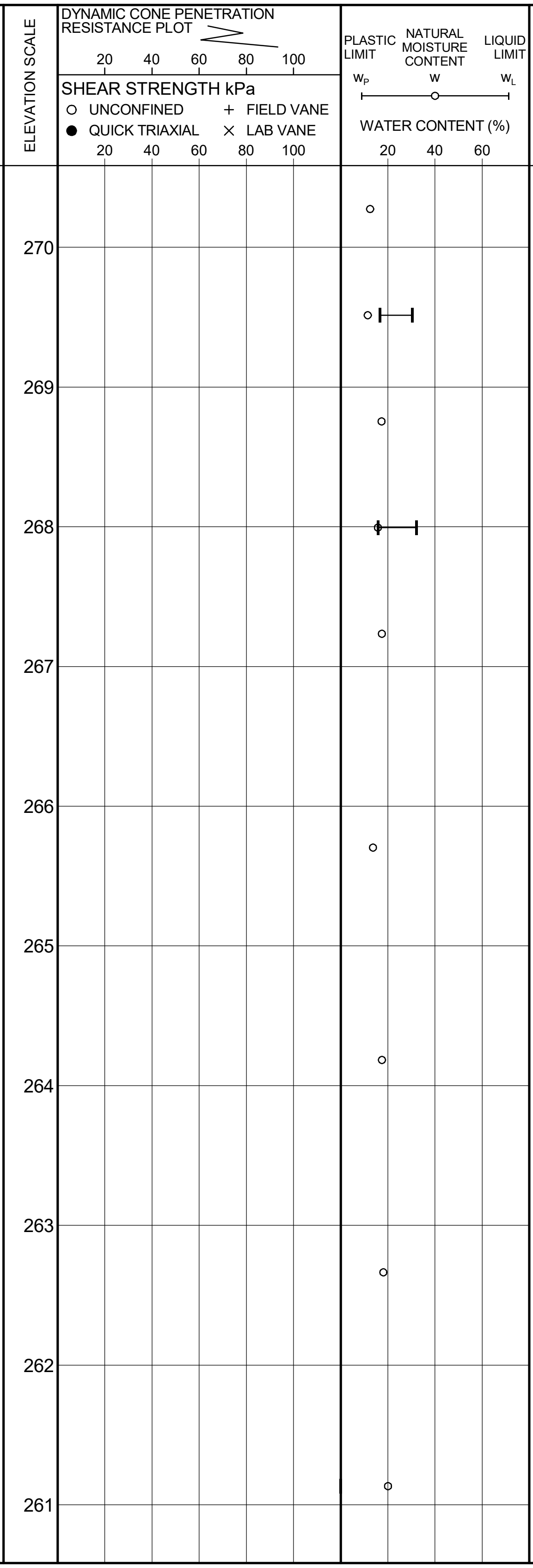
# RECORD OF BOREHOLE No PM-03

1 OF 4

**METRIC**

W.P. 3032-11-00
LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario
ORIGINATED BY JM
DIST West HWY 401
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY JM
DATUM Geodetic
DATE 2022.08.22 - 2022.08.24
LATITUDE 42.9340416
LONGITUDE -81.1916928
CHECKED BY GR

| ELEV<br>DEPTH | SOIL PROFILE<br>DESCRIPTION | SAMPLES<br>NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 270.6 | | | | | | | |
| 270.4<br>0.0 | 150mm TOPSOIL | | | | | | |
| 0.2 | FILL: SANDY CLAY to CLAYEY SILT, trace gravel, rootlets, and organics<br>Brown<br>Dry to moist<br>Firm to stiff | 1 | SS | 12 | | | PP=4.0<br>Su= 215 kPa |
| | | 2 | SS | 11 | | | 13 27 29 31 |
| | | 3 | SS | 7 | | | PP=3.25<br>Su= 174 kPa |
| 268.3<br>2.3 | CLAYEY SILT (CL), trace gravel and sand<br>Brown<br>Very stiff<br>Wet | 4 | SS | 14 | | | 2 14 37 48<br>PP=3.5<br>Su= 188 kPa |
| | | 5 | SS | 21 | | | PP=3.5<br>Su= 188 kPa |
| | Grey below 4.6 m | | | | ∇ | | |
| | | 6 | SS | 24 | | | PP=2.5<br>Su= 134 kPa |
| | | 7 | SS | 23 | | | PP=2.75<br>Su= 148 kPa |
| | | 8 | SS | 20 | | | PP=2.5<br>Su= 188 kPa |
| 261.4<br>9.1 | SANDY SILT to SILT (ML)<br>Grey<br>Compact to dense<br>Wet | 9 | SS | 31 | | | 0 16 79 5 |



Continued Next Page

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

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

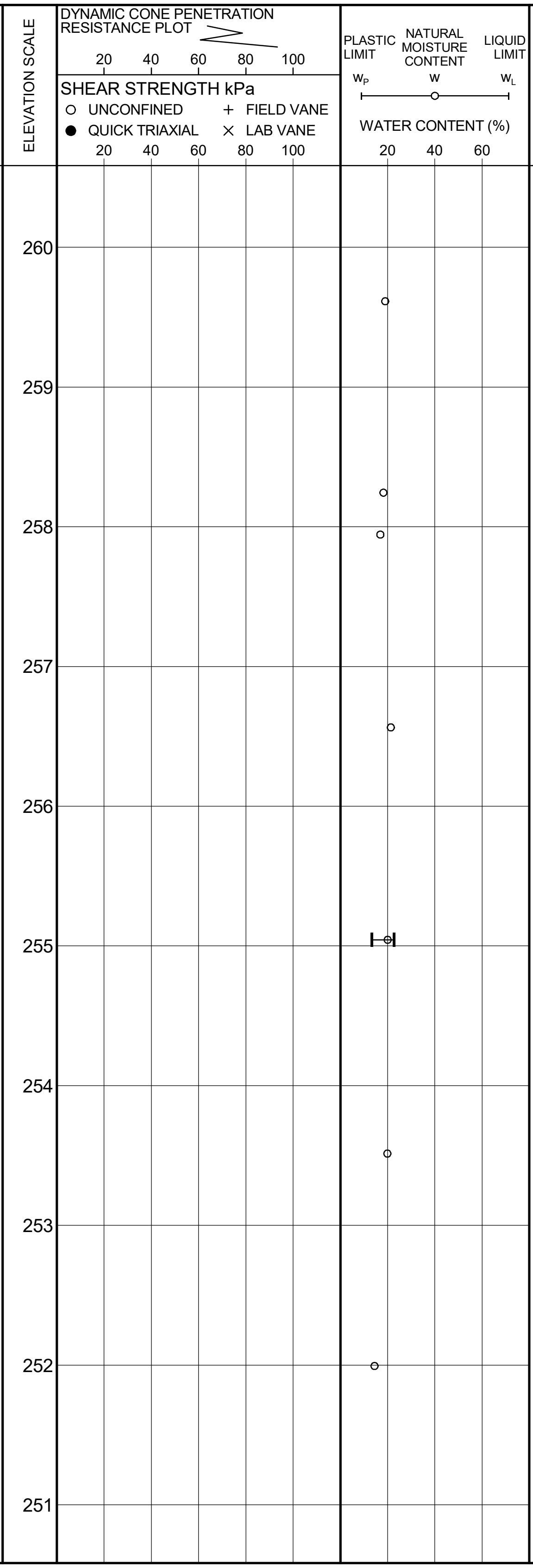
# RECORD OF BOREHOLE No PM-03

2 OF 4 **METRIC**

| | | | |
|---|---|---|---|
| W.P. 3032-11-00 | LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario | | ORIGINATED BY JM |
| DIST West HWY 401 | BOREHOLE TYPE Hollow Stem Augers | | COMPILED BY JM |
| DATUM Geodetic | DATE 2022.08.22 - 2022.08.24 | LATITUDE 42.9340416 LONGITUDE -81.1916928 | CHECKED BY GR |

| SOIL PROFILE ELEV / DEPTH | DESCRIPTION | STRAT PLOT |
|---|---|---|
| | SANDY SILT to SILT (ML)<br>Grey<br>Compact to dense<br>Wet *(continued)* | |
| 256.3<br>14.3 | SILTY CLAY (CL)<br>Grey<br>Moist to wet<br>Very stiff | |
| 252.4<br>18.2 | SILT (ML), trace sand<br>Grey<br>Wet<br>Dense to very dense | |

| SAMPLES NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|
| 10 | SS | 27 | | | | | | |
| 11 | SS | 46 | | | | | | |
| 12 | SS | 43 | | | | | | |
| 13 | SS | 28 | | | PP=2.0<br>Su= 107 kPa<br>0 | 0 | 58 | 42 |
| 14 | SS | 16 | | | PP=4.0<br>Su= 215 kPa | | | |
| 15 | SS | 46 | | | | | | |



Continued Next Page

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

# RECORD OF BOREHOLE No PM-03

3 OF 4 **METRIC**

| | | | |
|---|---|---|---|
| W.P. 3032-11-00 | LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario | | ORIGINATED BY JM |
| DIST West HWY 401 | BOREHOLE TYPE Hollow Stem Augers | | COMPILED BY JM |
| DATUM Geodetic | DATE 2022.08.22 - 2022.08.24 | LATITUDE 42.9340416 LONGITUDE -81.1916928 | CHECKED BY GR |

| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| ELEV / DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | | | GR | SA | SI | CL |
| | SILT (ML), trace sand<br>Grey<br>Wet<br>Dense to very dense *(continued)* | 16 | SS | 65 | | | | | | |
| | | 17 | SS | 36 | | | 0 | 13 | 66 | 20 |
| 248.6<br>21.9 | SILTY CLAY to CLAY (CL to CI), some sand, trace gravel, TILL<br>Grey<br>Moist to wet<br>Very stiff to hard | | | | | | | | | |
| | | 18 | SS | 38 | | | PP=3.5<br>Su= 188 kPa | | | |
| | | 19 | SS | 49 | | | PP=3.25<br>Su= 174 kPa | | | |
| | | 20 | SS | 26 | | | PP=2.75<br>Su=148 kPa | | | |
| | | 21 | SS | 34 | | | PP=2.25<br>Su= 121 kPa | | | |
| | | 22 | SS | 30 | | | 1 | 16 | 43 | 40 |
| | | | | | | | PP=2.5<br>Su= 134 kPa | | | |

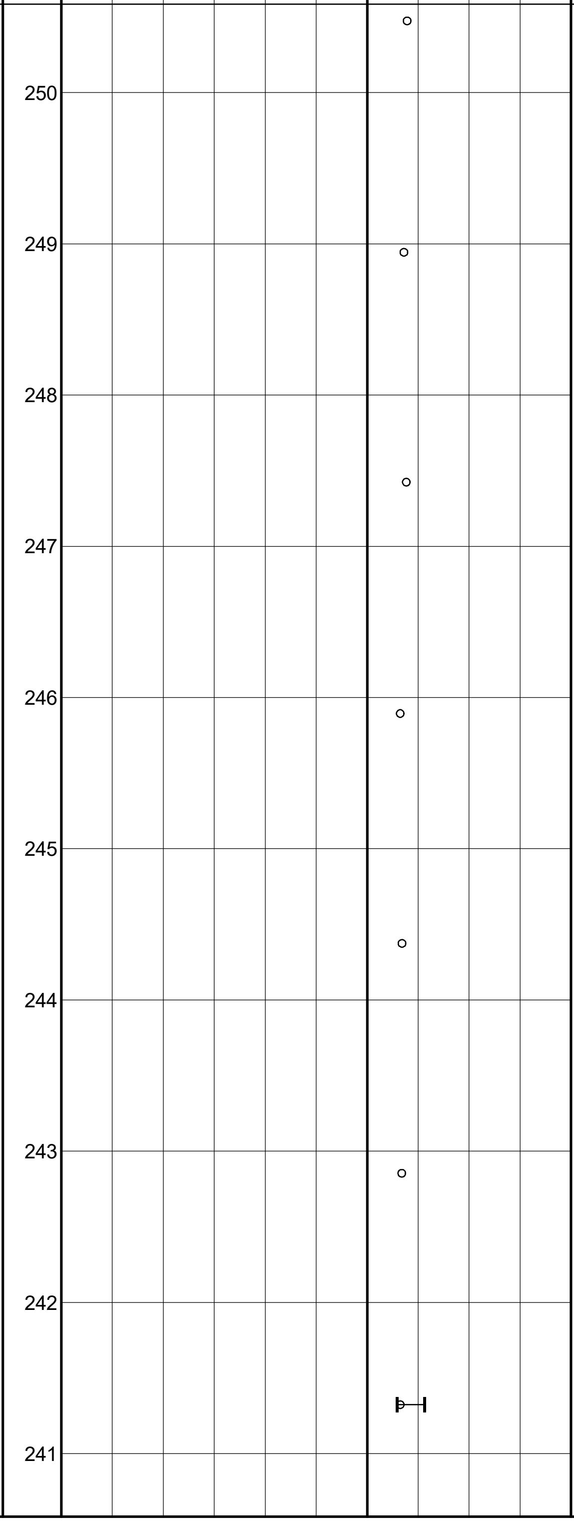
ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT $w_P$ NATURAL MOISTURE CONTENT $w$ LIQUID LIMIT $w_L$

WATER CONTENT (%) 20 40 60



Continued Next Page

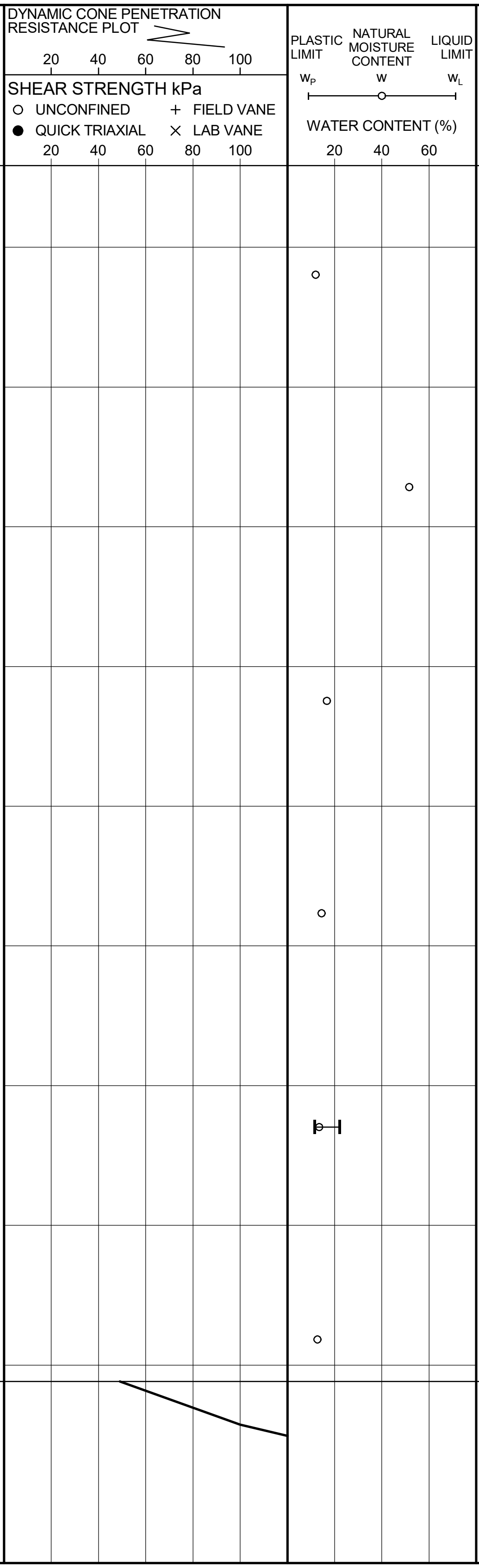
+³, ×³: Numbers refer to Sensitivity ○ ³% STRAIN AT FAILURE

# RECORD OF BOREHOLE No PM-03

4 OF 4 **METRIC**

W.P. 3032-11-00
LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario
ORIGINATED BY JM
DIST West HWY 401
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY JM
DATUM Geodetic
DATE 2022.08.22 - 2022.08.24
LATITUDE 42.9340416
LONGITUDE -81.1916928
CHECKED BY GR

| ELEV<br>DEPTH | SOIL PROFILE – DESCRIPTION | SAMPLES NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| | SILTY CLAY to CLAY (CL to CI), some sand, trace gravel, TILL<br>Grey<br>Moist to wet<br>Very stiff to hard *(continued)* | | | | | |
| | | | | | 240 | |
| | | 23 | SS | 26 | | PP=1.5<br>Su= 80 kPa |
| | | | | | 239 | |
| | Cobbles and gravel stones at 32m | | | | | |
| | | 24 | SS | 32 | | |
| | | | | | 238 | |
| | | | | | 237 | |
| | | 25 | SS | 35 | | PP=2.0<br>Su= 107 kPa |
| | | | | | 236 | |
| | | 26 | SS | 23 | | PP=1.25<br>Su= 67 kPa |
| | | | | | 235 | |
| | | | | | 234 | |
| | | 27 | SS | 21 | | 2 13 47 38<br>PP=1.25<br>Su= 67 kPa |
| | | | | | 233 | |
| | | 28 | SS | 30 | | PP=1.0<br>Su= 57 kPa |
| 231.9 | | | | | 232 | |
| 38.7 | END OF BOREHOLE<br><br>Groundwater and cave-in observation not made due to mud rotary drilling. Groundwater was inferred at 4.6 m depth.<br><br>Dynamic Cone Penetration Test from 38.7 m to 39.9 m.<br>DCPT refusal at 39.9m | | | | | |



GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m³

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

# RECORD OF BOREHOLE No PM-04

1 OF 2

**METRIC**

W.P. 3032-11-00
LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario
ORIGINATED BY JM
DIST West HWY 401
BOREHOLE TYPE Solid Stem Augers
COMPILED BY JM
DATUM Geodetic
DATE 2022.07.15 - 2022.07.15
LATITUDE 42.9338394
LONGITUDE -81.191269
CHECKED BY GR

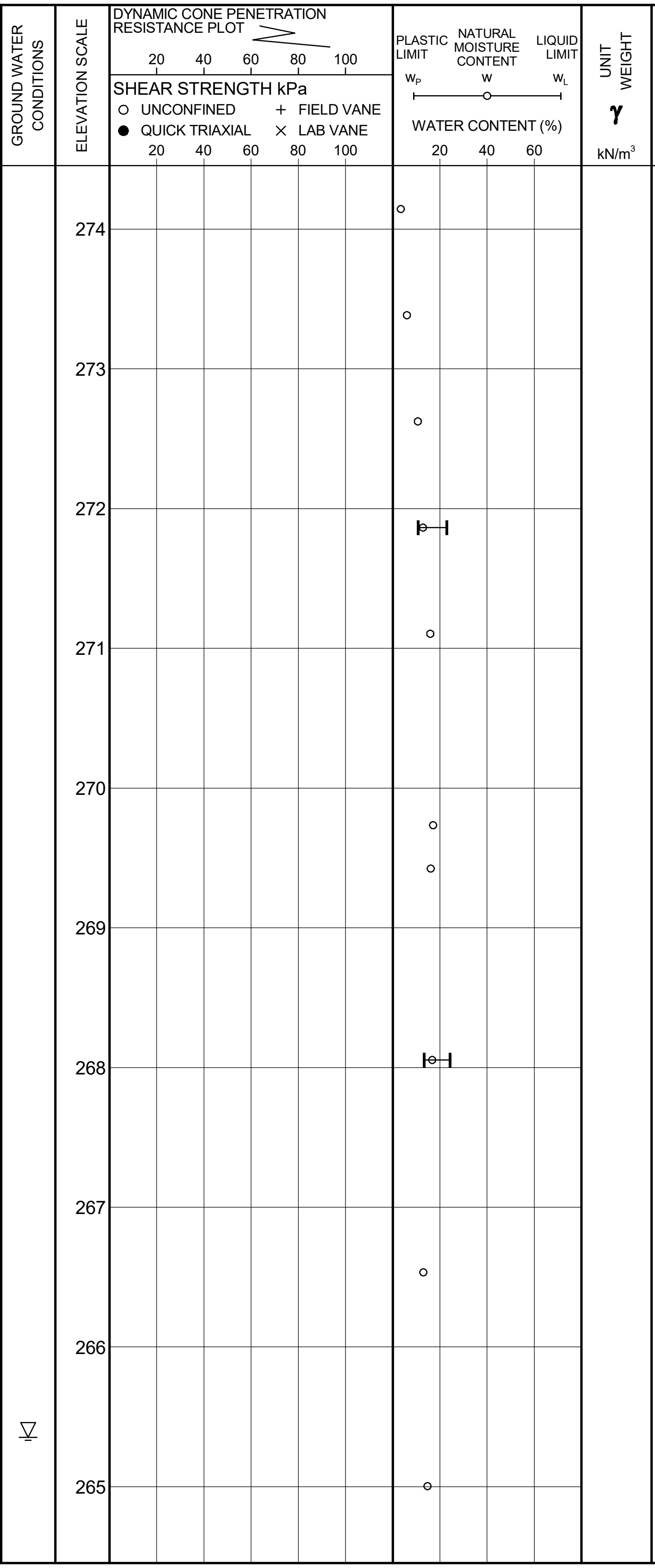
## SOIL PROFILE

| ELEV / DEPTH | DESCRIPTION |
|---|---|
| 274.5 | |
| 0.0 | 280 mm ASPHALT |
| 274.2<br>0.3 | FILL: SILTY SAND (SM), trace gravel. Contains construction debris.<br>Brown<br>Compact to dense<br>Moist |
| 272.9<br>1.5 | FILL: CLAYEY SILT (CL), trace sand and gravel<br>Brown<br>Stiff<br>Moist<br>SS5 contains trace rootlets<br>Sandy silt to silty sand layer at the top of SS6 |
| 268.9<br>5.6 | CLAY SILT (CL), trace sand and gravel<br>Brown<br>Very stiff<br>Moist |
| 267.3<br>7.2 | SILT with Sand (ML), trace clay<br>Brown to grey<br>Very loose to very dense<br>Moist to wet<br>Grey below 7.6 m |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|
| 1 | SS | 40 | |
| 2 | SS | 19 | |
| 3 | SS | 9 | |
| 4 | SS | 14 | Su > 100 kPa<br>2 32 33 33<br>PP=2.5<br>Su= 134 kPa |
| 5 | SS | 13 | |
| 6 | SS | 9 | Su > 100 kPa |
| 7 | SS | 16 | 1 8 51 40<br>PP=2.5<br>Su= 134 kPa |
| 8 | SS | 53 | |
| 9 | SS | 46 | |

GROUND WATER CONDITIONS: ▽ at approx. elev. 265.4



Continued Next Page

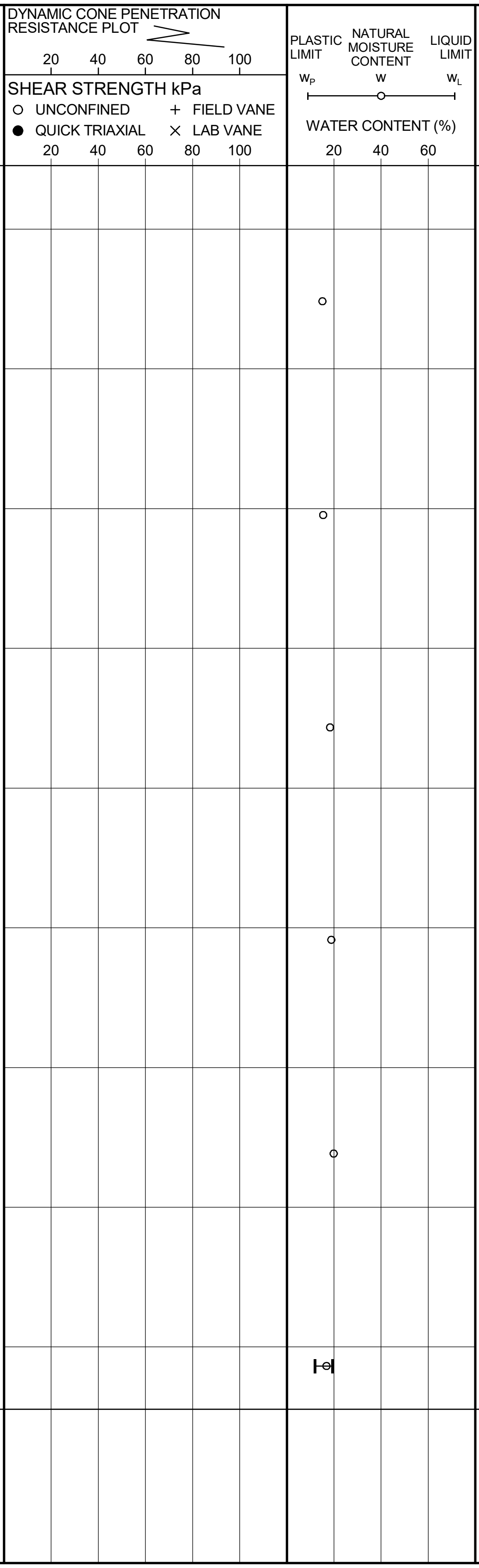
+³, ×³: Numbers refer to Sensitivity
○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No PM-04

2 OF 2 **METRIC**

| | | | |
|---|---|---|---|
| W.P. 3032-11-00 | LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario | | ORIGINATED BY JM |
| DIST West HWY 401 | BOREHOLE TYPE Solid Stem Augers | | COMPILED BY JM |
| DATUM Geodetic | DATE 2022.07.15 - 2022.07.15 | LATITUDE 42.9338394 LONGITUDE -81.191269 | CHECKED BY GR |

| ELEV / DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|
| | SILT with Sand (ML), trace clay<br>Brown to grey<br>Very loose to very dense<br>Moist to wet *(continued)* | | | | 264 | | | | |
| | | 10 | SS | 26 | 263 | | | | |
| | | 11 | SS | 18 | 262 | | | | |
| | Wet below 13.7 m | 12 | SS | 22 | 261<br>260 | | | | |
| | SS13 and SS14 very loose | 13 | SS | 0 | 259<br>258 | | | | |
| | | 14 | SS | 0 | 257 | 0 | 29 | 67 | 4 |
| 256.4<br>18.1 | CLAYEY SILT (CL)<br>Grey<br>Very stiff<br>Moist to wet | 15 | SS | 21 | 256 | 0 | 0 | 67 | 32 |
| 255.6<br>18.9 | END OF BOREHOLE<br><br>Groundwater level and cave-in measured at approximately 9.1 m and 13.4 m below grade, respectively; in open borehole. | | | | | | | | |



Columns: STRAT PLOT, GROUND WATER CONDITIONS, UNIT WEIGHT γ kN/m³, REMARKS & GRAIN SIZE DISTRIBUTION (%)

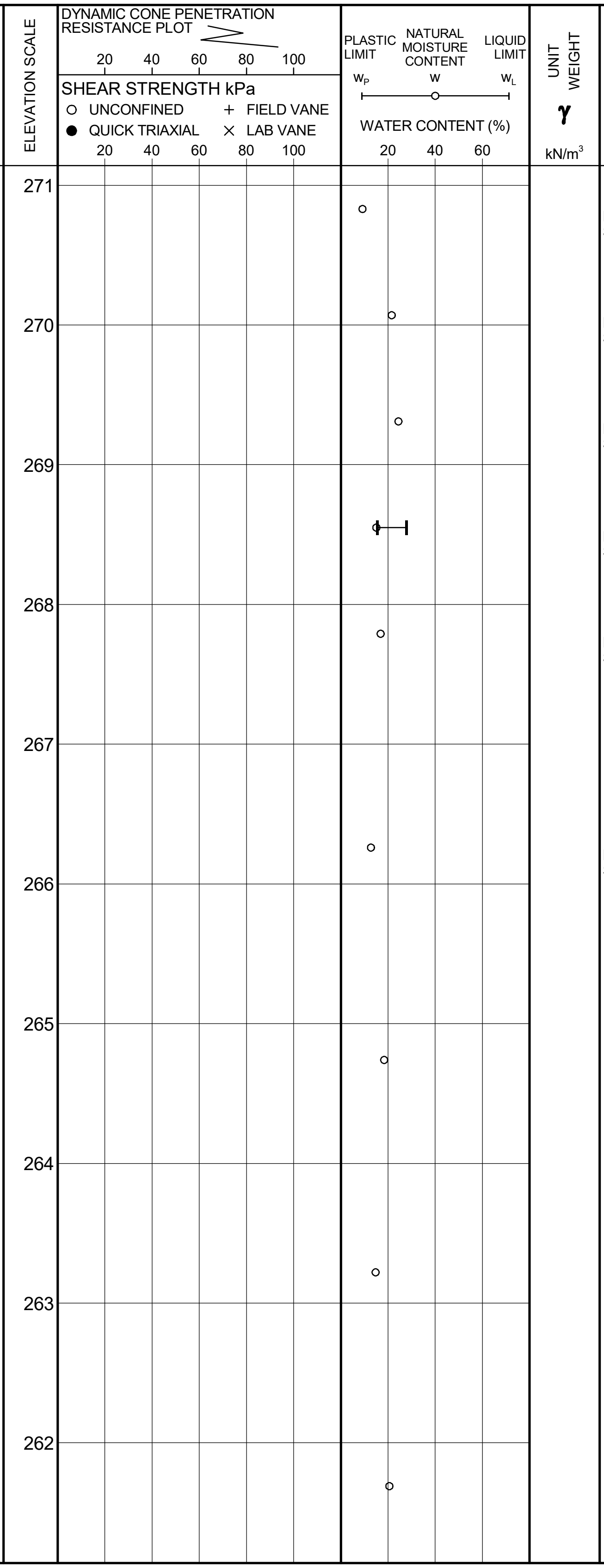
+ ³, × ³: Numbers refer to Sensitivity ○ ³% STRAIN AT FAILURE

# RECORD OF BOREHOLE No PM-05

1 OF 2 **METRIC**

| | | | |
|---|---|---|---|
| W.P. 3032-11-00 | LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario | | ORIGINATED BY JM |
| DIST West HWY 401 | BOREHOLE TYPE Hollow Stem Augers | | COMPILED BY JM |
| DATUM Geodetic | DATE 2022.08.24 - 2022.08.24 | LATITUDE 42.9340869 LONGITUDE -81.1915827 | CHECKED BY GR |

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|
| 271.1<br>0.0 | 100 mm TOPSOIL | | | | |
| 271.0<br>0.1 | FILL: CLAYEY SILT (CL), trace sand, gravel and rootlets<br>Brown<br>Stiff<br>Moist | 1 | SS | 16 | PP=3.25<br>Su= 174 kPa |
| | | 2 | SS | 11 | PP=3.0<br>Su= 161 kPa |
| | | 3 | SS | 8 | PP=1.75<br>Su= 94 kPa |
| 268.9<br>2.2 | CLAYEY SILT (CL), some sand, trace gravel<br>Brown<br>Stiff to very stiff<br>Moist to wet<br>SS4 contains rock fragments<br><br>Grey below 3 m | 4 | SS | 16 | 2 13 43 41<br>PP=3.0<br>Su= 161 kPa |
| | | 5 | SS | 13 | PP=2.75<br>Su= 148 kPa |
| | | 6 | SS | 14 | PP=4.0<br>Su= 215 kPa |
| | | 7 | SS | 17 | |
| | | 8 | SS | 16 | |
| 262.4<br>8.7 | SILT (ML), trace to some clay<br>Grey<br>Loose to compact<br>Wet | 9 | SS | 5 | 0 9 69 22 |



Continued Next Page

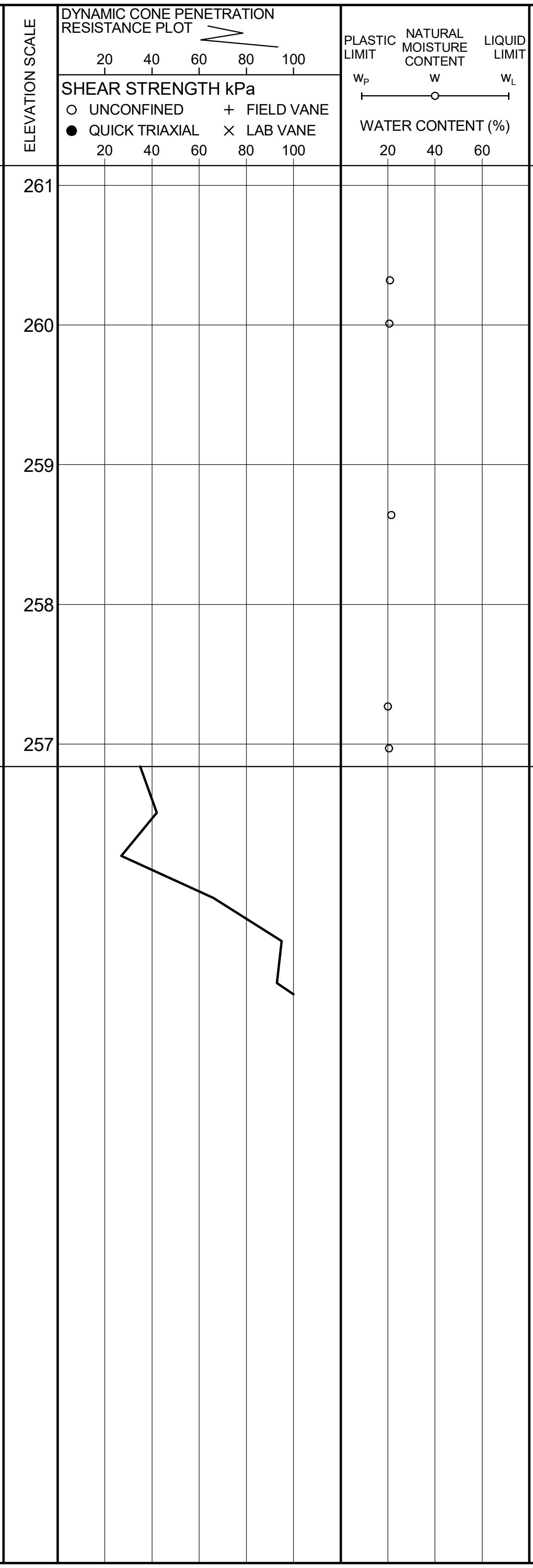
$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

# RECORD OF BOREHOLE No PM-05

2 OF 2 **METRIC**

W.P. 3032-11-00
LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario
ORIGINATED BY JM
DIST West HWY 401
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY JM
DATUM Geodetic
DATE 2022.08.24 - 2022.08.24 LATITUDE 42.9340869 LONGITUDE -81.1915827
CHECKED BY GR

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | STRAT PLOT | SAMPLES NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | UNIT WEIGHT γ kN/m³ | REMARKS | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | SILT (ML), trace to some clay<br>Grey<br>Loose to compact<br>Wet *(continued)* | | | | | | | | | | | |
| | SS10 contains a 50 mm silty clay seam | | 10 | SS | 7 | | | | | | | |
| | | | 11 | SS | 10 | | | | 0 | 9 | 85 | 6 |
| 257.0<br>14.1 | | | 12 | SS | 27 | | | | | | | |
| 256.8<br>14.3 | SILTY CLAY<br>Grey<br>Wet | | | | | | | PP=4.0<br>Su= 215 kPa | | | | |
| | END OF BOREHOLE<br><br>Borehole open and dry upon completion fo drilling.<br><br>Dynamic Cone Penetration Test from approximately 14.3 m to 16 m. | | | | | | | | | | | |



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

# RECORD OF BOREHOLE No PM-06

1 OF 2 **METRIC**

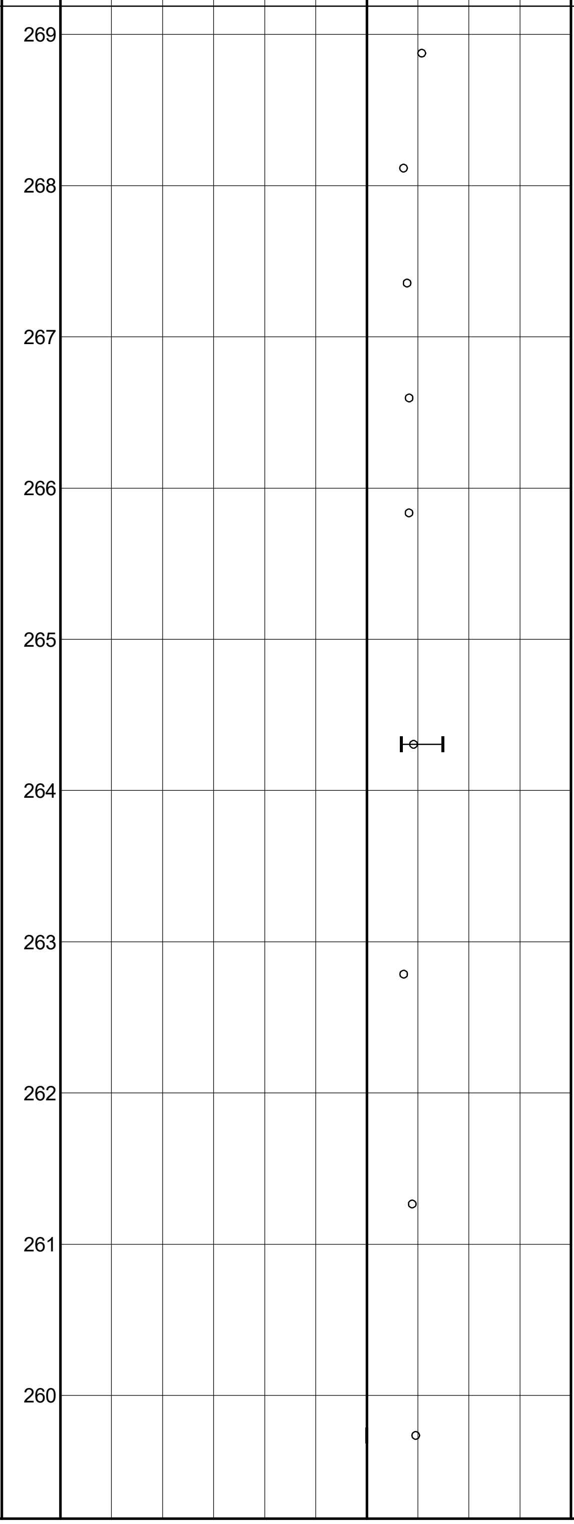
W.P. 3032-11-00
LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario
ORIGINATED BY BD
DIST West HWY 401
BOREHOLE TYPE Solid Stem Augers
COMPILED BY JM
DATUM Geodetic
DATE 2022.08.19 - 2022.08.19 LATITUDE 42.9331649 LONGITUDE -81.1921059
CHECKED BY GR

| ELEV<br>DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
|---|---|---|---|---|---|---|
| 269.2 | | | | | | |
| 0.0 | 250 mm TOPSOIL | | | | | |
| 268.9<br>0.3 | FILL: SILTY SAND, trace gravel and rootlets<br>Brown<br>Very Loose<br>Moist | 1 | SS | 3 | | |
| 268.4<br>0.8 | SILTY CLAY (CL), trace gravel and sand<br>Brown<br>Stiff to very stiff<br>Moist | 2 | SS | 12 | | |
| | | 3 | SS | 15 | | PP>4.5<br>Su> 241 kPa |
| | | 4 | SS | 17 | | PP= 4.0<br>Su= 215 kPa |
| | | 5 | SS | 10 | | PP= 4.25<br>Su= 228 kPa |
| | | | | | ∇ | |
| | Grey below 4.6 m | 6 | SS | 8 | | 6 8 37 50<br>PP= 3.0<br>Su= 161 kPa |
| 263.1<br>6.1 | SILT (ML), some clay, trace sand<br>Grey<br>Compact<br>Moist to wet | 7 | SS | 26 | | |
| | | 8 | SS | 22 | | |
| | | 9 | SS | 19 | | 0 6 72 22<br>Non-Plastic |

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 (%)
20 40 60

UNIT WEIGHT γ kN/m³



Continued Next Page

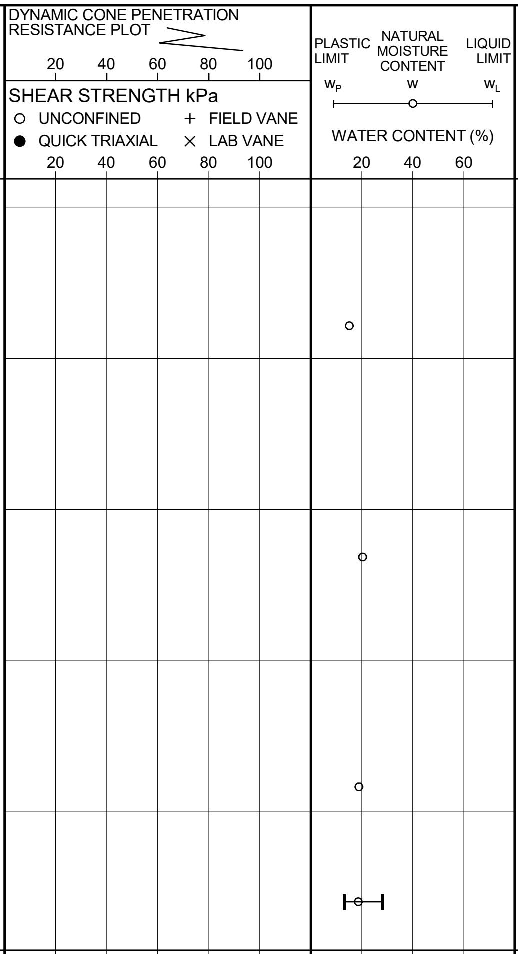
+³, ×³: Numbers refer to Sensitivity   ○³% STRAIN AT FAILURE

# RECORD OF BOREHOLE No PM-06

2 OF 2 **METRIC**

W.P. 3032-11-00
LOCATION Pond Mills Road Overhead/ Highbury, London, Ontario
ORIGINATED BY BD
DIST West HWY 401
BOREHOLE TYPE Solid Stem Augers
COMPILED BY JM
DATUM Geodetic
DATE 2022.08.19 - 2022.08.19 LATITUDE 42.9331649 LONGITUDE -81.1921059
CHECKED BY GR

| SOIL PROFILE | | | SAMPLES | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV<br>DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR | SA | SI | CL |
| | SILT (ML), some clay, trace sand<br>Grey<br>Compact<br>Moist to wet *(continued)* | | | | | | 259 | | | | | |
| | | | 10 | SS | 25 | | 258 | | | | | |
| | | | 11 | SS | 13 | | 257 | | | | | |
| | | | | | | | 256 | | | | | |
| 255.5<br>13.7 | SILTY CLAY (CL)<br>Grey<br>Stiff to very stiff<br>Wet | | 12 | SS | 17 | | 255 | | | | | |
| | | | 13 | SS | 14 | | | | 0<br>PP=2.25<br>Su= 121 kPa | 0 | 45 | 55 |
| 254.1<br>15.1 | END OF BOREHOLE<br><br>Groundwater level and cave-in measured at approximately 6.7 m below grade upon completion of drilling. | | | | | | | | | | | |



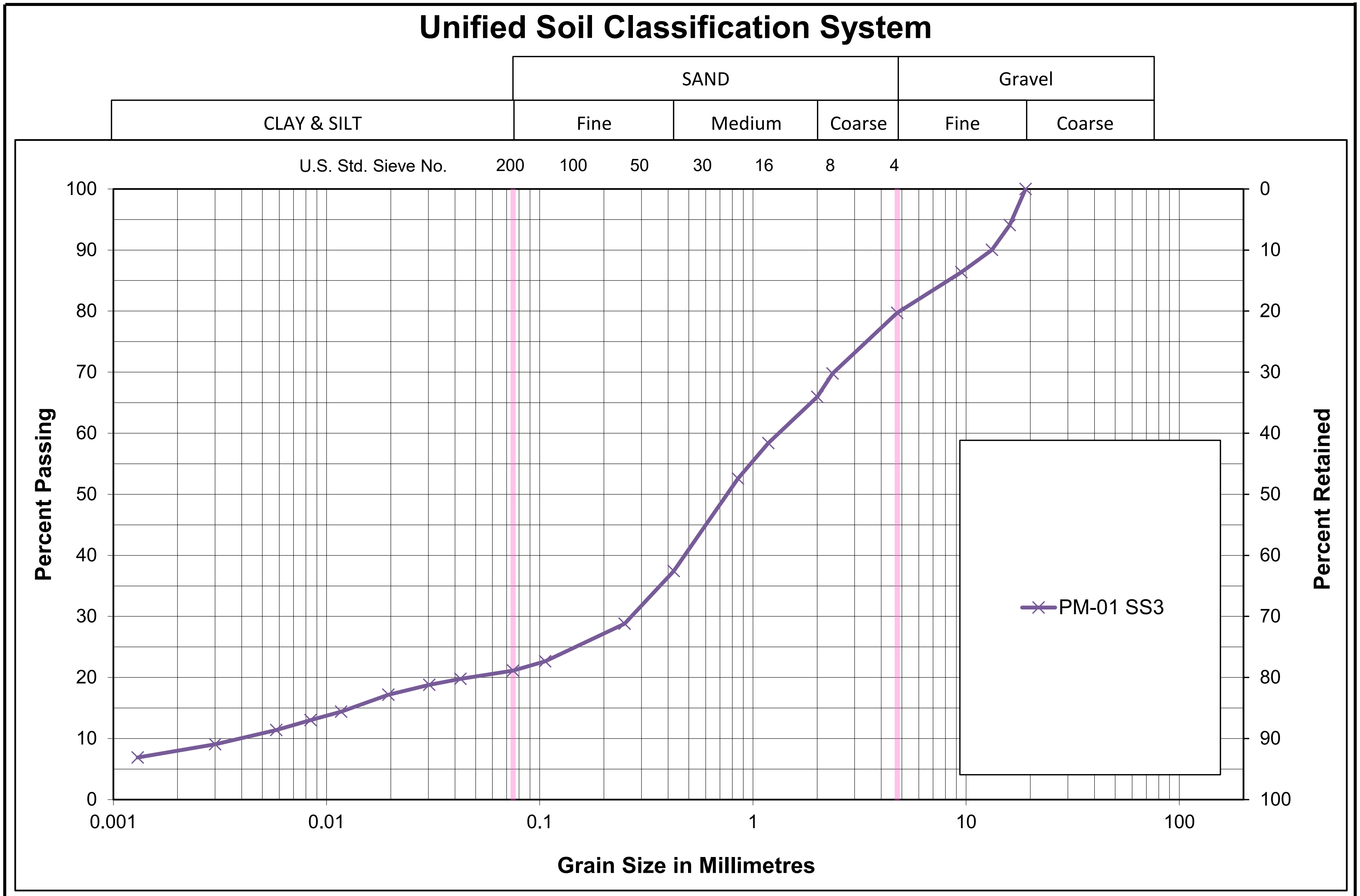
+³, ×³: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

# APPENDIX D

## D.1 LABORATORY TEST RESULTS - FIGURES D1 – D11: GRAIN SIZE DISTRIBUTION PLOTS AND PLASTICITY CHARTS

## D.2 CHEMICAL TESTING LABORATORY TEST RESULTS



# GRAIN SIZE DISTRIBUTION

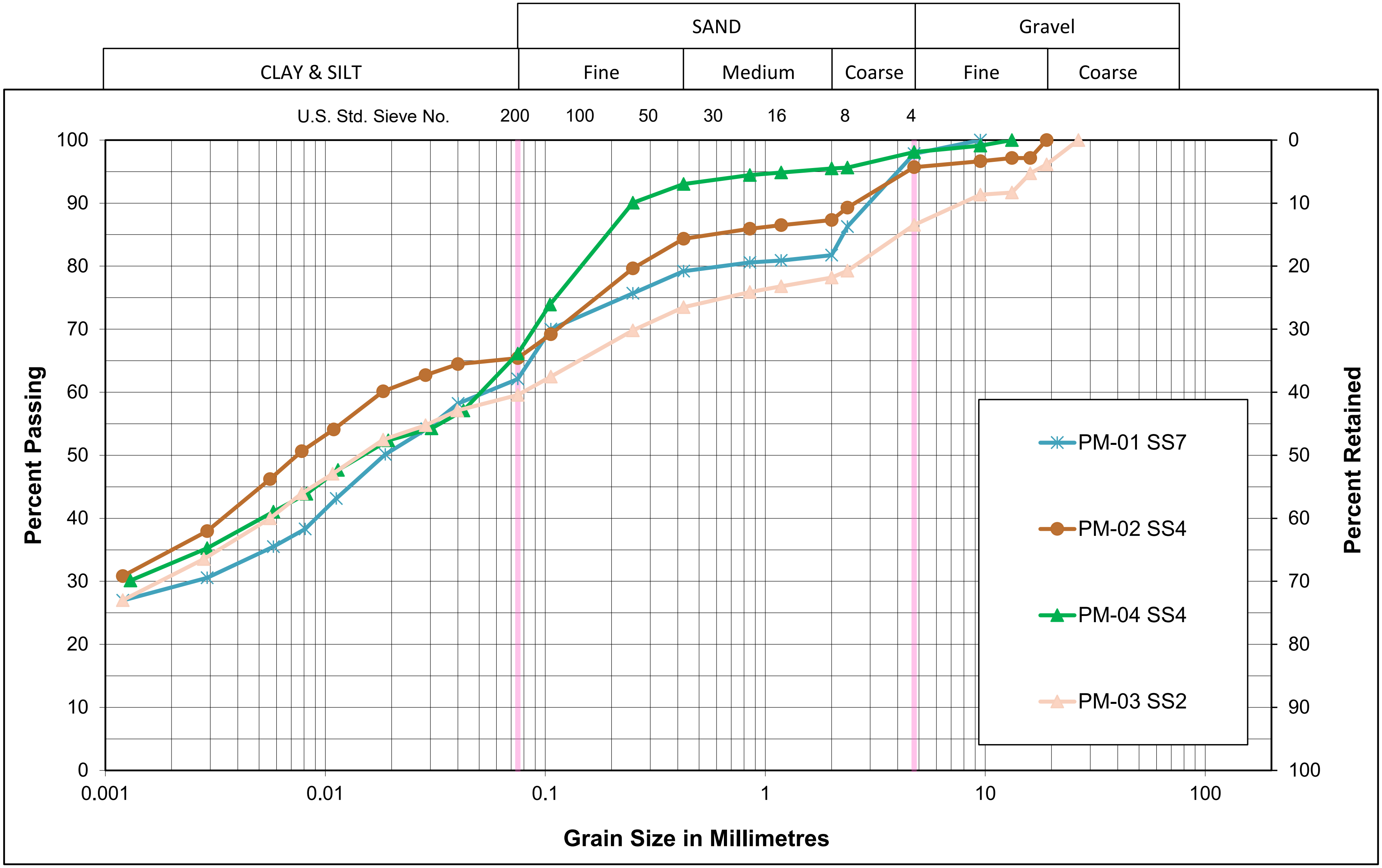
Granular Fill: Silty SAND with Gravel (SM)

Pond Mills Road Overpass Replacement- Highway 401

Figure No. D1

Project No. 165001239

## Unified Soil Classification System



Stantec

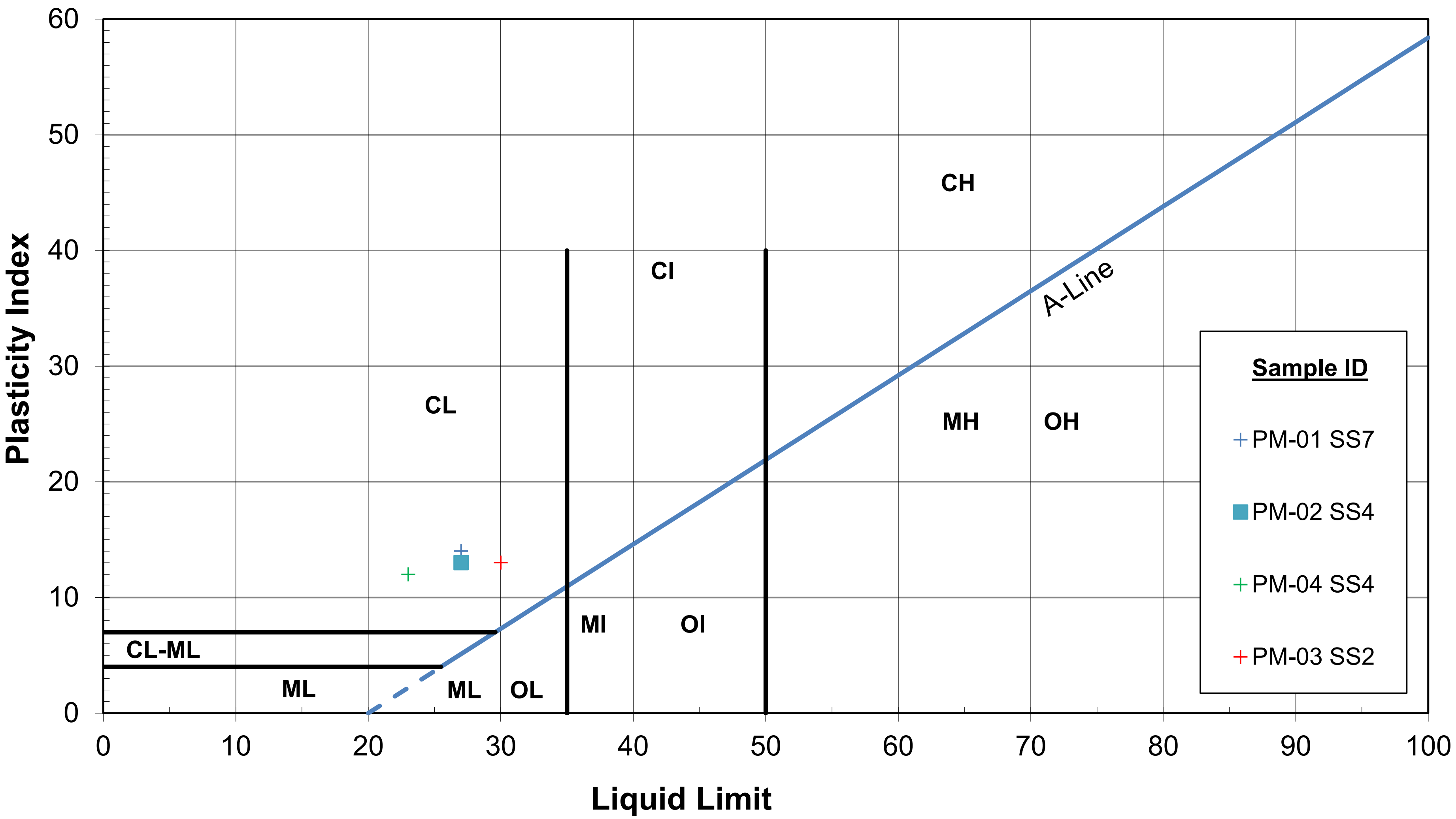
# GRAIN SIZE DISTRIBUTION

Cohesive Fill: Clayey Silt/Sandy Clayey Silt (CL)

Pond Mills Road Overpass Replacement- Highway 401

Figure No. D2

Project No. 165001239



Stantec

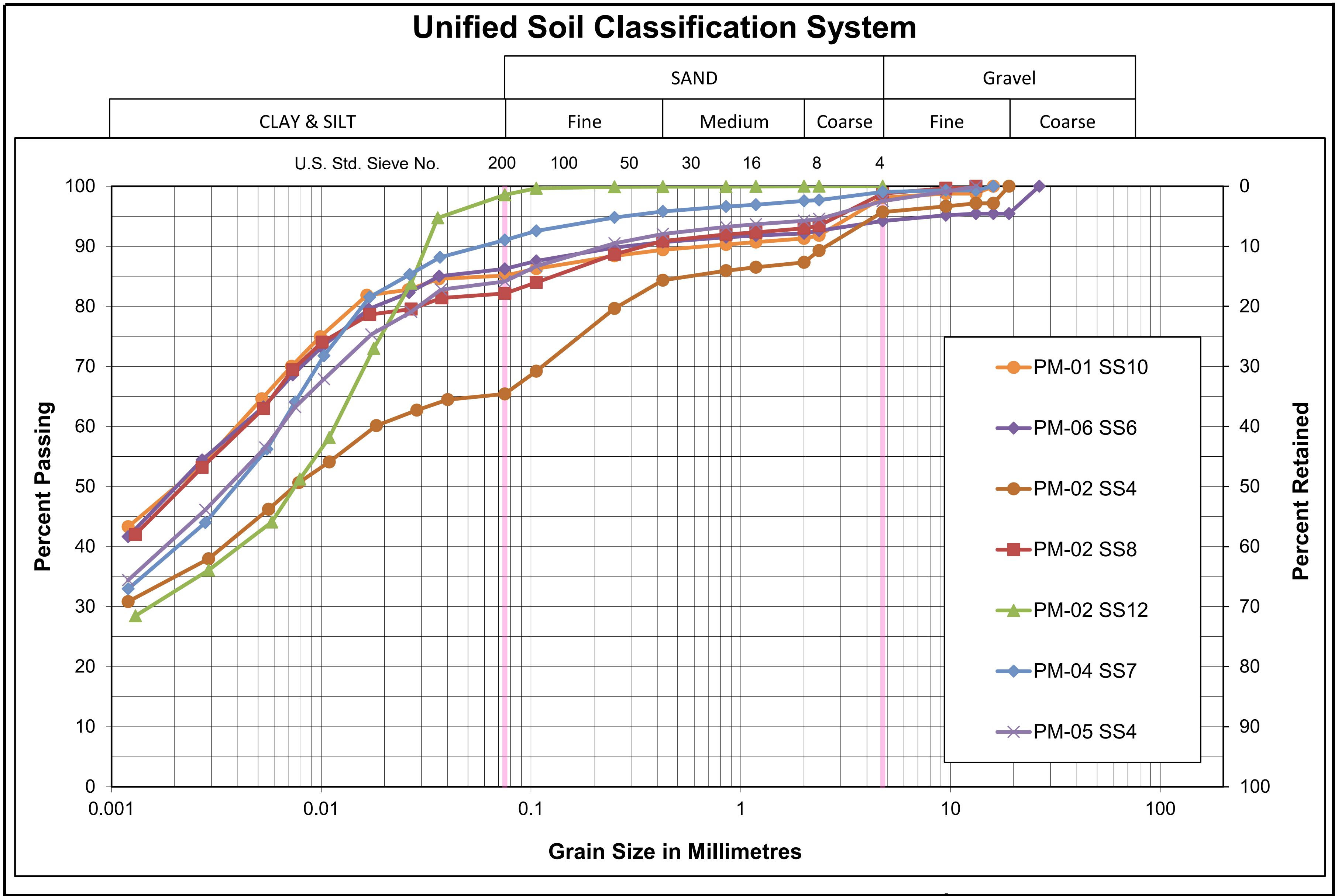
Cohesive Fill: Clayey Silt/Sandy Clayey Silt (CL)

Pond Mills Road Overpass Replacement- Highway 401

# PLASTICITY CHART

Figure No. D3

Project No. 165001239



Stantec

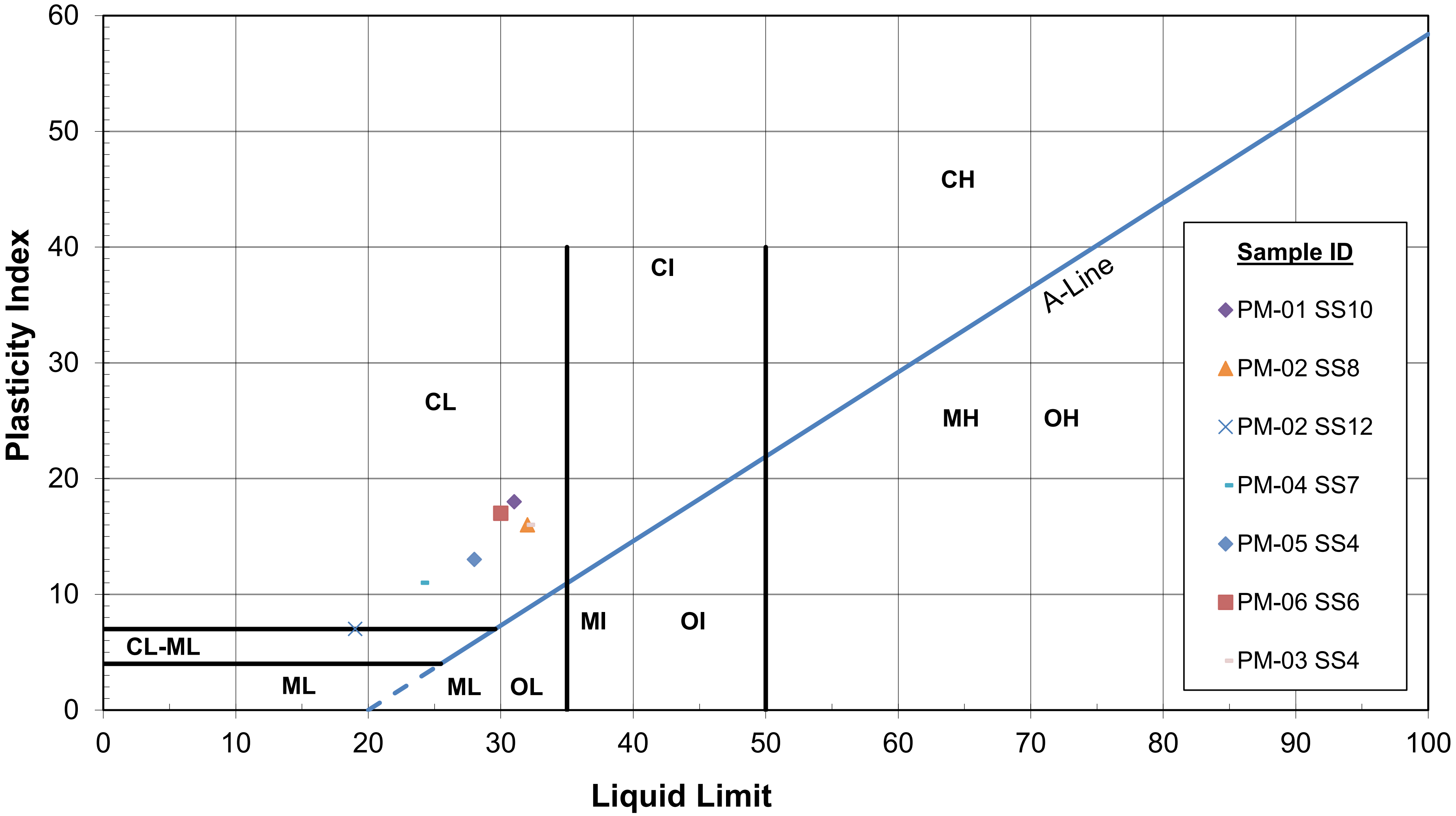
# GRAIN SIZE DISTRIBUTION

## Upper Clayey Silt to Silty Clay (CL)

Pond Mills Road Overpass Replacement- Highway 401

Figure No. D4

Project No. 165001239



Stantec

Upper Clayey Silt to Silty Clay (CL)

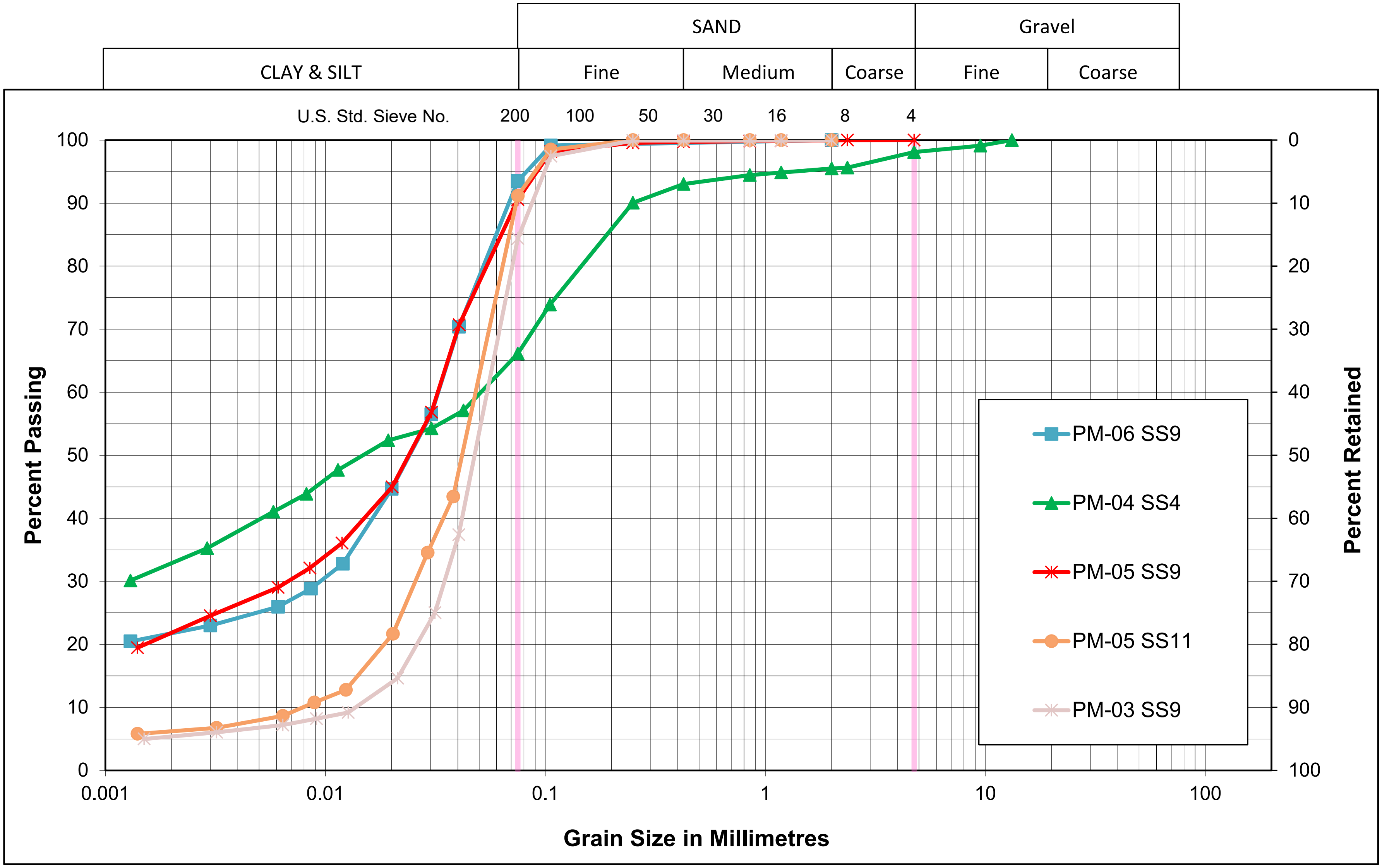
Pond Mills Road Overpass Replacement- Highway 401

# PLASTICITY CHART

Figure No. D5

Project No. 165001239

# Unified Soil Classification System



Stantec

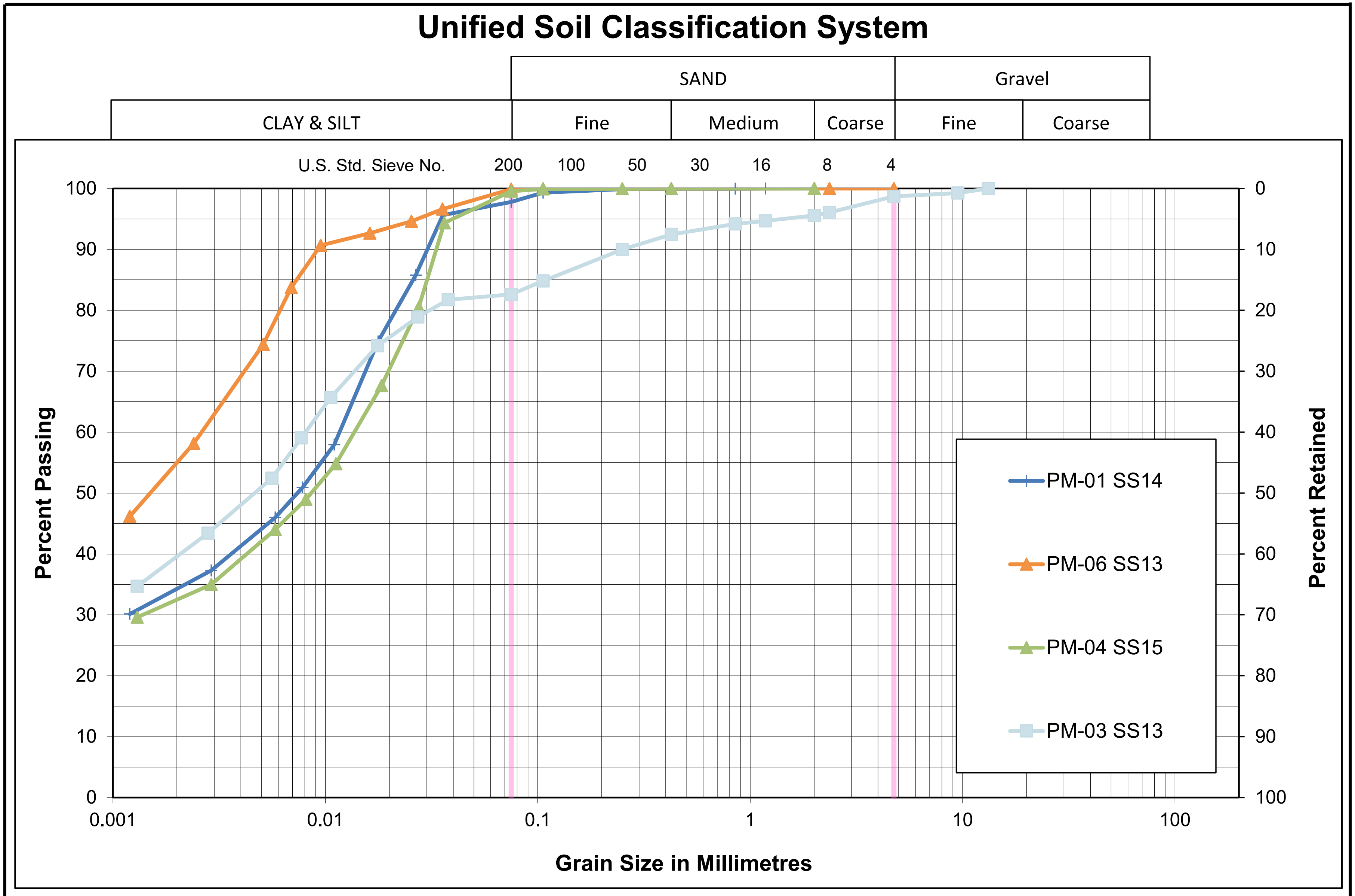
# GRAIN SIZE DISTRIBUTION

## Upper Silt to Sandy Silt (ML)

Pond Mills Road Overpass Replacement- Highway 401

Figure No. D6

Project No. 165001239



Stantec

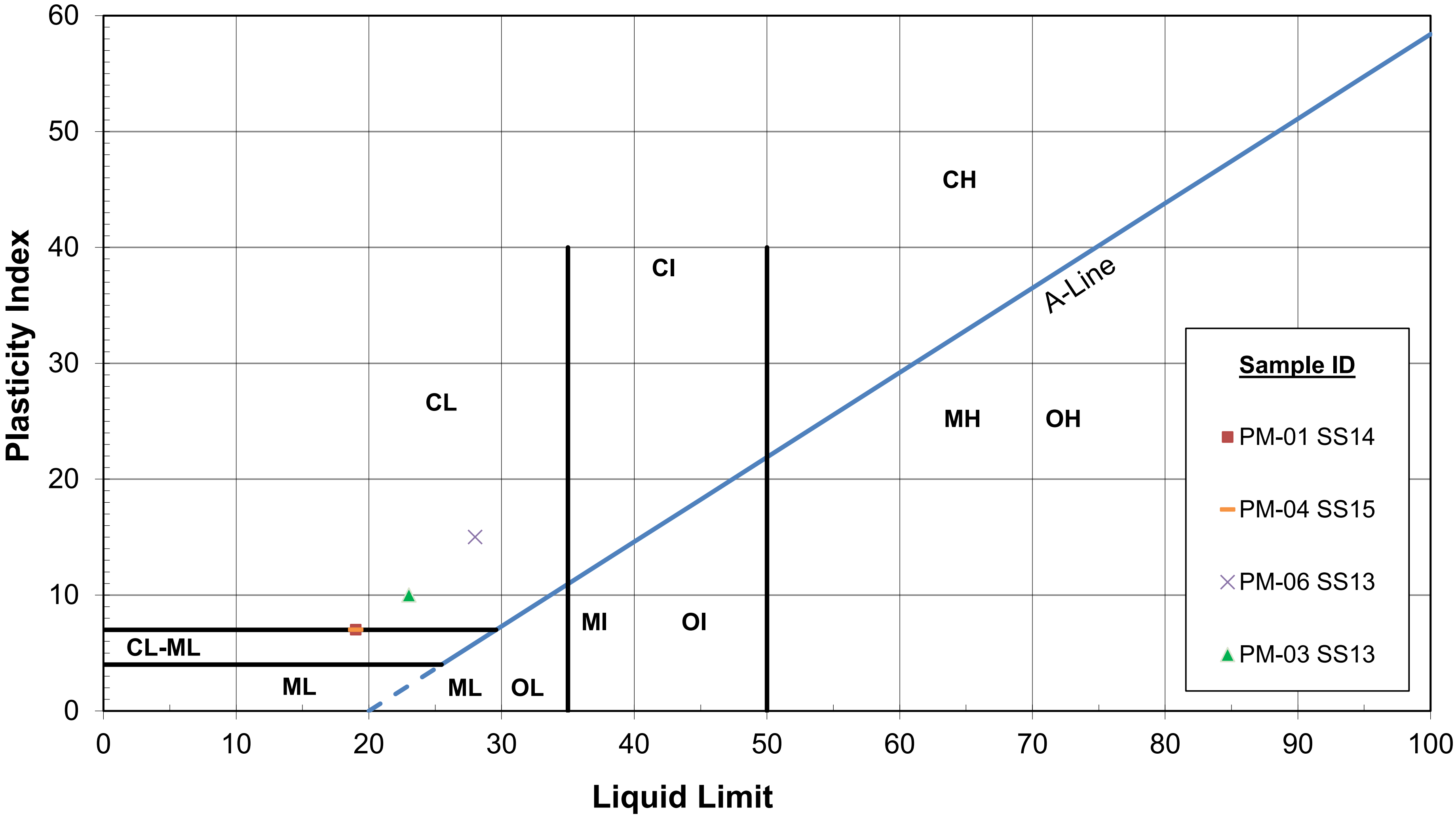
# GRAIN SIZE DISTRIBUTION

## Lower Clayey Silt (CL)

Pond Mills Road Overpass Replacement- Highway 401

Figure No. D7

Project No. 165001239



Stantec

Lower Clayey Silt (CL)

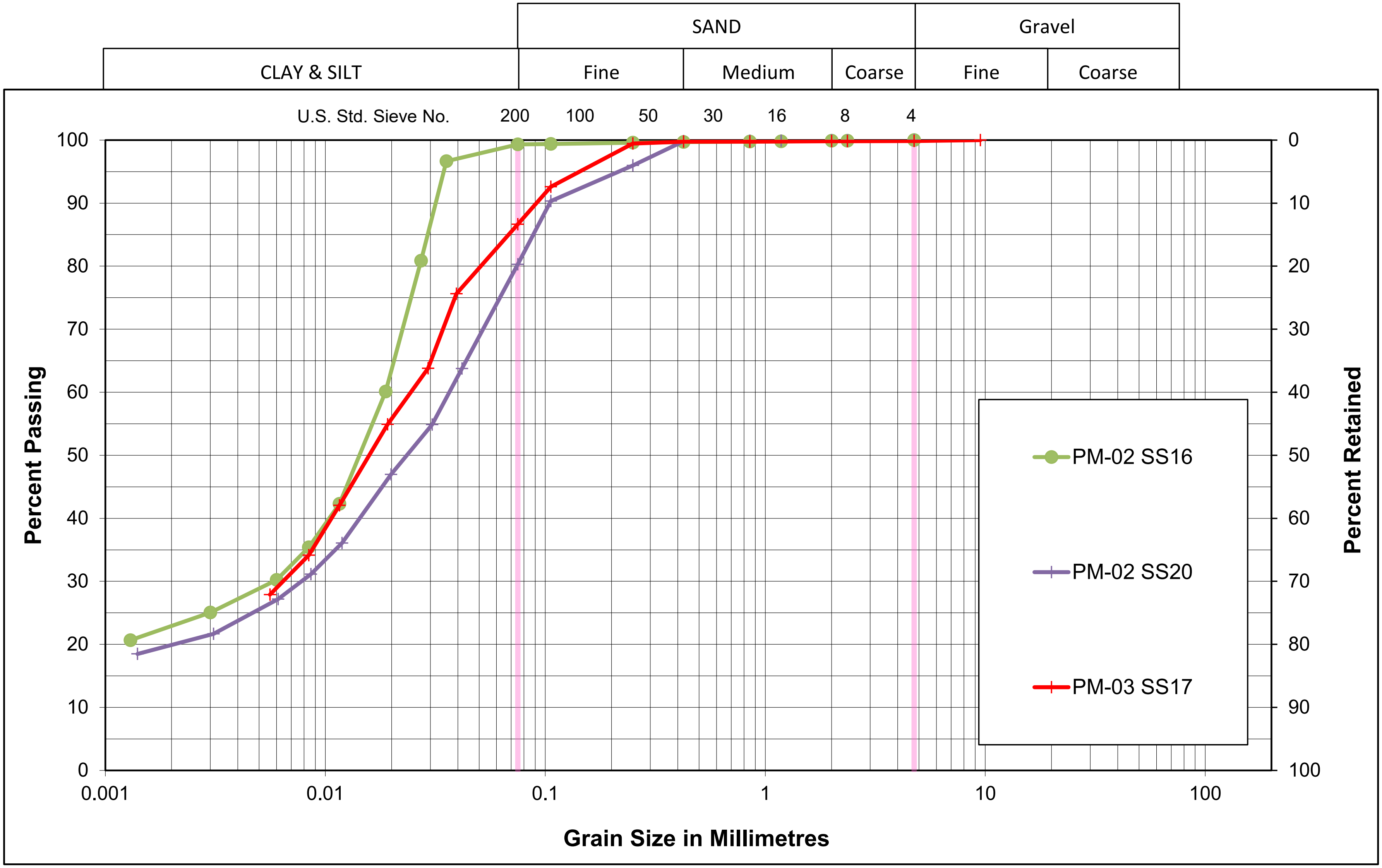
Pond Mills Road Overpass Replacement- Highway 401

# PLASTICITY CHART

Figure No. D8

Project No. 165001239

## Unified Soil Classification System



Stantec

# GRAIN SIZE DISTRIBUTION

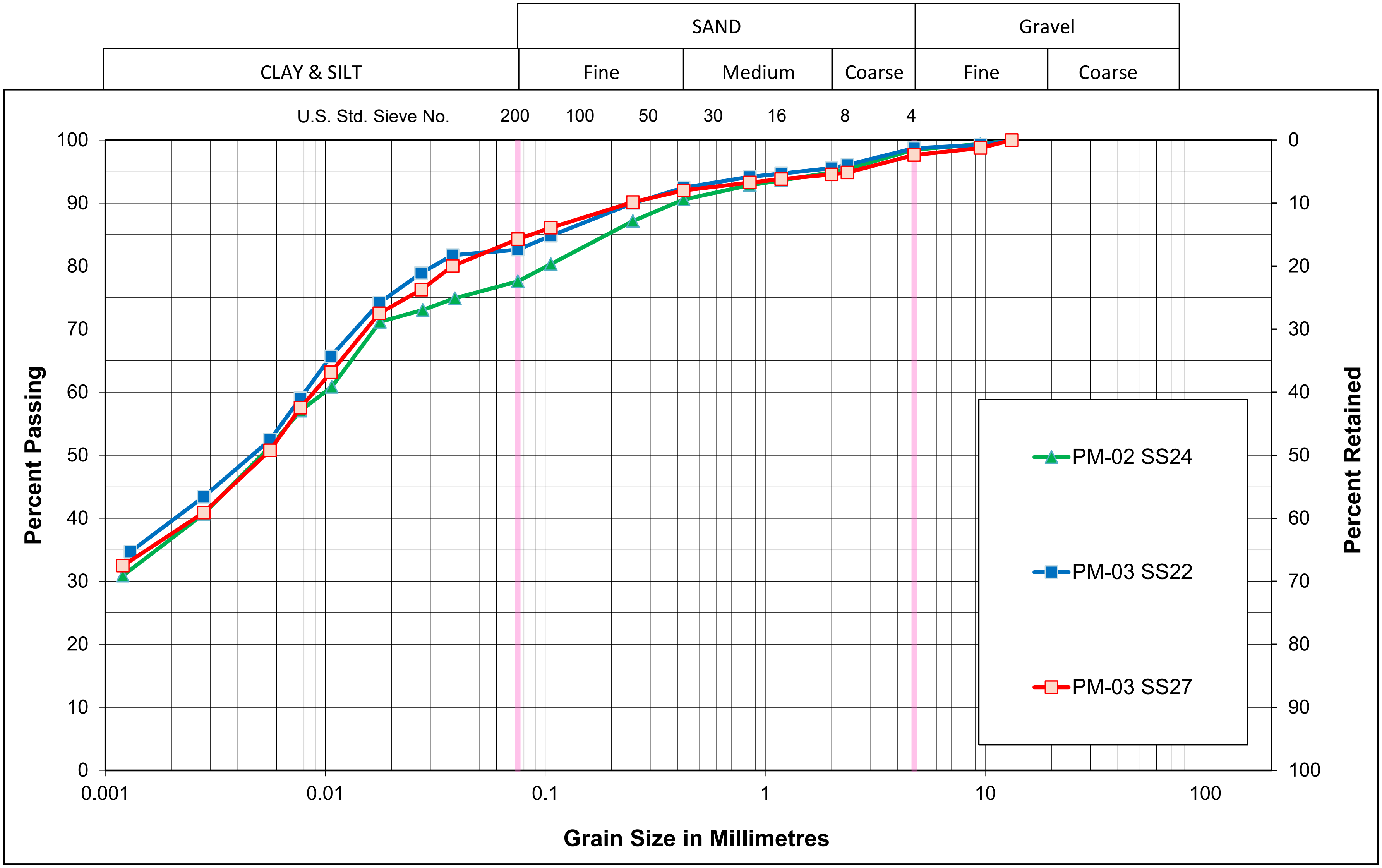
## Lower Silt and Silt with Sand (ML)

Pond Mills Road Overpass Replacement- Highway 401

Figure No. D9

Project No. 165001239

# Unified Soil Classification System



Stantec

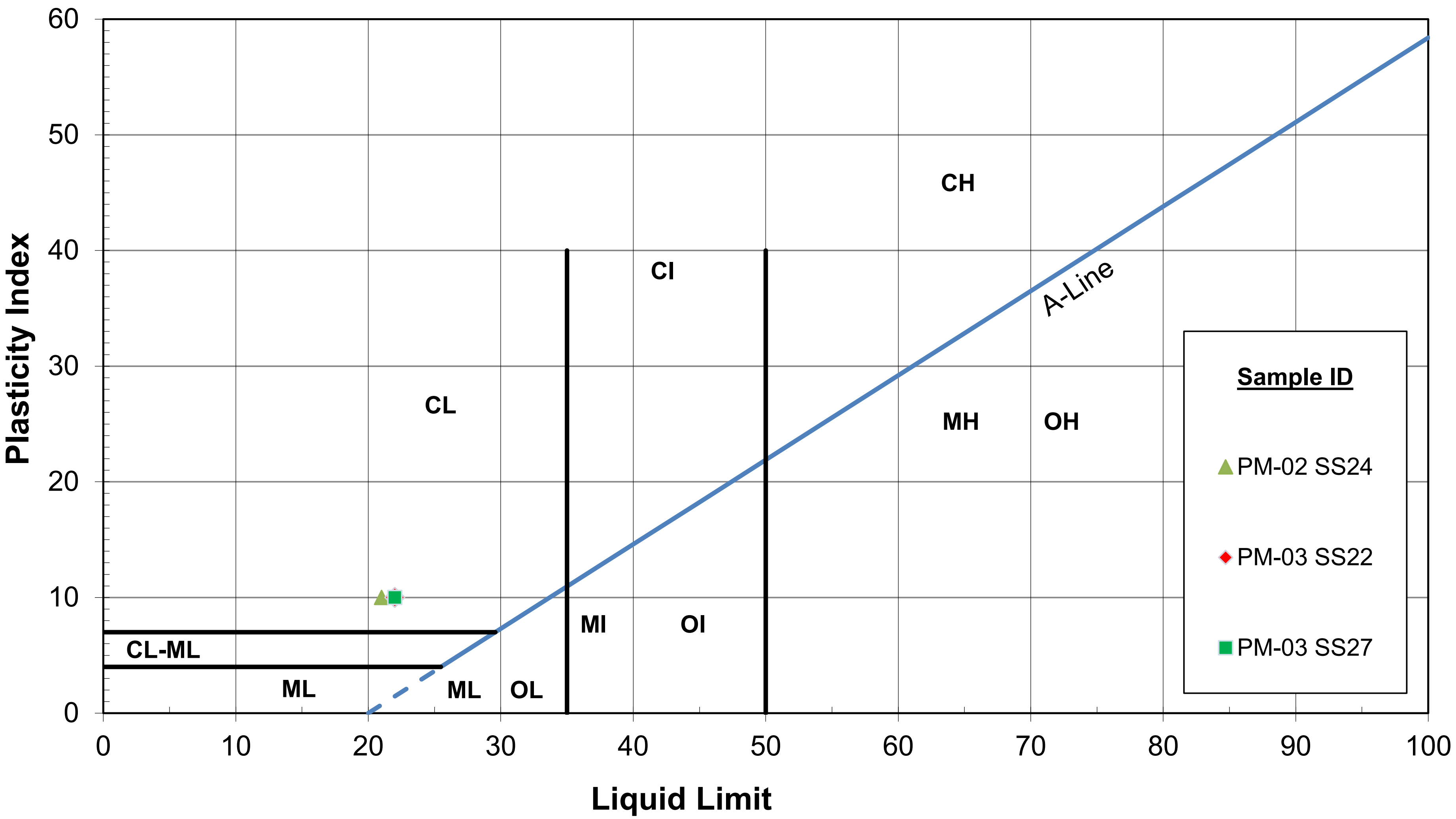
# GRAIN SIZE DISTRIBUTION

## Clayey Silt Till (CL)

Pond Mills Road Overpass Replacement- Highway 401

Figure No. D10

Project No. 165001239



Stantec

Clayey Silt Till (CL)

Pond Mills Road Overpass Replacement- Highway 401

# PLASTICITY CHART

Figure No. D11

Project No. 165001239

AGAT Laboratories

2910 12TH STREET NE
CALGARY, ALBERTA
CANADA T2E 7P7
TEL (403)735-2005
FAX (403)735-2771
http://www.agatlabs.com

# Certificate of Analysis

AGAT WORK ORDER: 22T944869
PROJECT: 165001239.651

CLIENT NAME: STANTEC CONSULTING LTD

ATTENTION TO: Amoldeep Gill

## (283-042) Sulfide (CGY)

DATE SAMPLED: Sep 12, 2022 DATE RECEIVED: Sep 14, 2022 DATE REPORTED: Sep 22, 2022 SAMPLE TYPE: Soil

| | Analyte: | Sulfide |
|---|---|---|
| | Unit: | % |
| Sample ID (AGAT ID) | RDL: | 0.01 |
| (MC-01) - SS8 (4302866) | | 0.02 |
| (S-06-3) - SS8 (4302868) | | 0.06 |
| (S-08-1) - SS8 (4302869) | | 0.05 |
| (PM-03-2) - SS8 (4302870) | | 0.07 |
| (PM-02-1) - SS6 (4302871) | | <0.01 |
| (S-02) - SS6 (4302872) | | 0.01 |
| (S-07) - SS8 (4302873) | | 0.01 |
| (EL-02-1) - SS6 (4302874) | | 0.05 |
| (MC-02) - SS8 (4302875) | | 0.07 |
| (MS-01) - SS4 (4302881) | | 0.03 |

Comments: RDL - Reported Detection Limit

Analysis performed at AGAT Calgary (unless marked by *)
Insufficient Sample : IS
Sample Not Received : SNR

Certified By: M. White

AGAT Laboratories

# Certificate of Analysis

AGAT WORK ORDER: 22T944869
PROJECT: 165001239.651

2910 12TH STREET NE
CALGARY, ALBERTA
CANADA T2E 7P7
TEL (403)735-2005
FAX (403)735-2771
http://www.agatlabs.com

CLIENT NAME: STANTEC CONSULTING LTD

ATTENTION TO: Amoldeep Gill

## Corrosivity Package

DATE SAMPLED: Sep 12, 2022 DATE RECEIVED: Sep 14, 2022 DATE REPORTED: Sep 22, 2022 SAMPLE TYPE: Soil

|  | Analyte: | Chloride (2:1) | Sulphate (2:1) | pH (2:1) | Electrical Conductivity (2:1) | Resistivity (2:1) (Calculated) | Redox Potential 1 | Redox Potential 2 | Redox Potential 3 |
|---|---|---|---|---|---|---|---|---|---|
|  | Unit: | µg/g | µg/g | pH Units | mS/cm | ohm.cm | mV | mV | mV |
| Sample ID (AGAT ID) | RDL: | 2 | 2 | NA | 0.005 | 1 | NA | NA | NA |
| (MC-01) - SS8 (4302866) |  | 470 | 97 | 6.68 | 0.916 | 1090 | 417 | 417 | 416 |
| (S-06-3) - SS8 (4302868) |  | 89 | 120 | 6.65 | 0.390 | 2560 | 415 | 415 | 415 |
| (S-08-1) - SS8 (4302869) |  | 199 | 98 | 6.81 | 0.571 | 1750 | 343 | 348 | 349 |
| (PM-03-2) - SS8 (4302870) |  | 8 | 96 | 6.79 | 0.221 | 4520 | 321 | 323 | 324 |
| (PM-02-1) - SS6 (4302871) |  | 206 | 16 | 6.62 | 0.471 | 2120 | 295 | 304 | 309 |
| (S-02) - SS6 (4302872) |  | 486 | 62 | 7.31 | 0.990 | 1010 | 257 | 265 | 274 |
| (S-07) - SS8 (4302873) |  | 1090 | 35 | 7.09 | 2.09 | 478 | 317 | 317 | 317 |
| (EL-02-1) - SS6 (4302874) |  | 1290 | 155 | 7.38 | 2.66 | 376 | 202 | 211 | 207 |
| (MC-02) - SS8 (4302875) |  | 287 | 403 | 6.66 | 0.920 | 1090 | 216 | 226 | 233 |
| (MS-01) - SS4 (4302881) |  | 296 | 29 | 7.45 | 0.687 | 1460 | 243 | 249 | 248 |

Comments: RDL - Reported Detection Limit

4302866-4302881 EC, pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter.
Redox potential measured on as received sample. Due to the potential for rapid change in sample equilibrium chemistry with exposure to oxidative/reduction conditions laboratory results may differ from field measured results.
Redox potential measurement in soil is quite variable and non reproducible due in part, to the general heterogeneity of a given soil. It is also related to the introduction of increased oxygen into the sample after extraction. The interpretation of soil redox potential should be considered in terms of its general range rather than as an absolute measurement.

Analysis performed at AGAT Toronto (unless marked by *)
Insufficient Sample : IS
Sample Not Received : SNR

ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO
CHARTERED
NIVINE BASILY
CHEMIST

Certified By: Nivine Basily

**AGAT Laboratories**

5835 Coopers Avenue
Mississauga, Ontario
L4Z 1Y2
www.agatlabs.com · webearth.agatlabs.com

# Chain of Custody Record

Ph.: 905.712.5100 · Fax: 905.712.5122 · Toll Free: 800.856.6261

**Laboratory Use Only**

Arrival Temperature: 20.8/20.9/21.4.
AGAT WO #. 22T944869
Lab Temperature:
Notes: 1 box / No Ice

## Client Information:

Company: Stantec Consulting Ltd.
Contact: Amoldeep Gill
Address: 300-675 Cochran Drive West Tower
Phone: 905-479-9345 Fax: 905-944-9889
Project: 165001239.651 PO:
AGAT Quotation #:

**Please note, if quotation number is not provided, client will be billed full price for analysis.**

## Regulatory Requirements:

☐ Regulation 153/09 *(reg. 511 Amend.)*
Table ______ *Indicate one*
☐ Ind/Com
☐ Res/Park
☐ Agriculture
Soil Texture *(check one)*
☐ Coarse ☐ Fine

☐ Sewer Use
Region ______ *Indicate one*
☐ Sanitary
☐ Storm

☐ Regulation 558
☐ CCME
☐ Other *(specify)*
☐ Prov. Water Quality Objectives (PWQO)
☐ None

## Turnaround Time Required (TAT) Required*

**Regular TAT**
☑ 5 to 7 Working Days

**Rush TAT** (please provide prior notification)
**Rush Surcharges Apply**
☐ 3 Working Days
☐ 2 Working Days
☐ 1 Working Day

**OR**

Date Required (Rush surcharges may apply):

**TAT is exclusive of weekends and statutory holidays*

## Invoice To:

Same: Yes ☑ No ☐
Company:
Contact:
Address:

**Is this a drinking water sample?**
(potable water intended for human consumption)
☐ Yes ☐ No
If "Yes", please use the ***Drinking Water Chain of Custody Form***

Is this submission for a **Record of Site Condition?**
☐ Yes ☐ No

## Legend Matrix

| | | | |
|---|---|---|---|
| **GW** | Ground Water | **O** | Oil |
| **SW** | Surface Water | **P** | Paint |
| **SD** | Sediment | **S** | Soil |

## Report Information – reports to be sent to:

1. Name: Amoldeep Gill
Email: amoldeep.gill@stantec.com
2. Name: Gwangha Roh
Email: gwangha.roh@stantec.com

| Sample Identification | Date Sampled | Time Sampled | Sample Matrix | # of Containers | Comments Site/Sample Information |
|---|---|---|---|---|---|
| (MC-01) - SS8 | 12-Sep-22 | | | 1 | 17.5' |
| (S-06-3) - SS8 | 12-Sep-22 | | | 1 | 17.5' |
| (S-08-1) - SS8 | 12-Sep-22 | | | 1 | 17.5' |
| (PM-03-2) - SS8 | 12-Sep-22 | | | 1 | 25' |
| (PM-02-1) - SS6 | 12-Sep-22 | | | 1 | 15' |
| (S-02) - SS6 | 12-Sep-22 | | | 1 | 12.5' |
| (S-07) - SS8 | 12-Sep-22 | | | 1 | 17.5' |
| (EL-02-1) - SS6 | 12-Sep-22 | | | 1 | 12.5' |
| (MC-02) - SS8 | 12-Sep-22 | | | 1 | 17.5' |
| (MS-01) - SS4 | 12-Sep-22 | | | 1 | 7.5' |
| | | | | | |
| | | | | | |

| Sample Identification | Metals and Inorganics | Metal Scan | Hydride Forming Metals | Client Custom Metals | ORPs: ☐ B-HWS ☐ Cl- ☐ CN-<br>☐ EC ☐ FOC ☐ Cr+6- ☐ SAR<br>☐ $NO_3/NO_2$ ☐ N-Total ☐ Hg ☐ pH | Nutients: ☐ TP ☐ $NH_3$ ☐ TKN<br>☐ $NO_3$ ☐ $NO_2$ ☐ $NO_3/NO_3$ | VOC: ☐ VOC ☐ THM ☐ BTEX | CCME Fractions 1 to 4 | ABNs | PAHs | Chlorophenols | PCBs | Organochlorine Pesticides | TCLP Metals/Inorganics | TCLP: | Sewer Use | Corrosivity Pckg (pH, Redox Potential, | sulphates and sulphides contents, | chlorides contents and resistivity) | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| (MC-01) - SS8 | | | | | | | | | | | | | | | | | X | X | X | | | | | |
| (S-06-3) - SS8 | | | | | | | | | | | | | | | | | X | X | X | | | | | |
| (S-08-1) - SS8 | | | | | | | | | | | | | | | | | X | X | X | | | | | |
| (PM-03-2) - SS8 | | | | | | | | | | | | | | | | | X | X | X | | | | | |
| (PM-02-1) - SS6 | | | | | | | | | | | | | | | | | X | X | X | | | | | |
| (S-02) - SS6 | | | | | | | | | | | | | | | | | X | X | X | | | | | |
| (S-07) - SS8 | | | | | | | | | | | | | | | | | X | X | X | | | | | |
| (EL-02-1) - SS6 | | | | | | | | | | | | | | | | | X | X | X | | | | | |
| (MC-02) - SS8 | | | | | | | | | | | | | | | | | X | X | X | | | | | |
| (MS-01) - SS4 | | | | | | | | | | | | | | | | | X | X | X | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | |

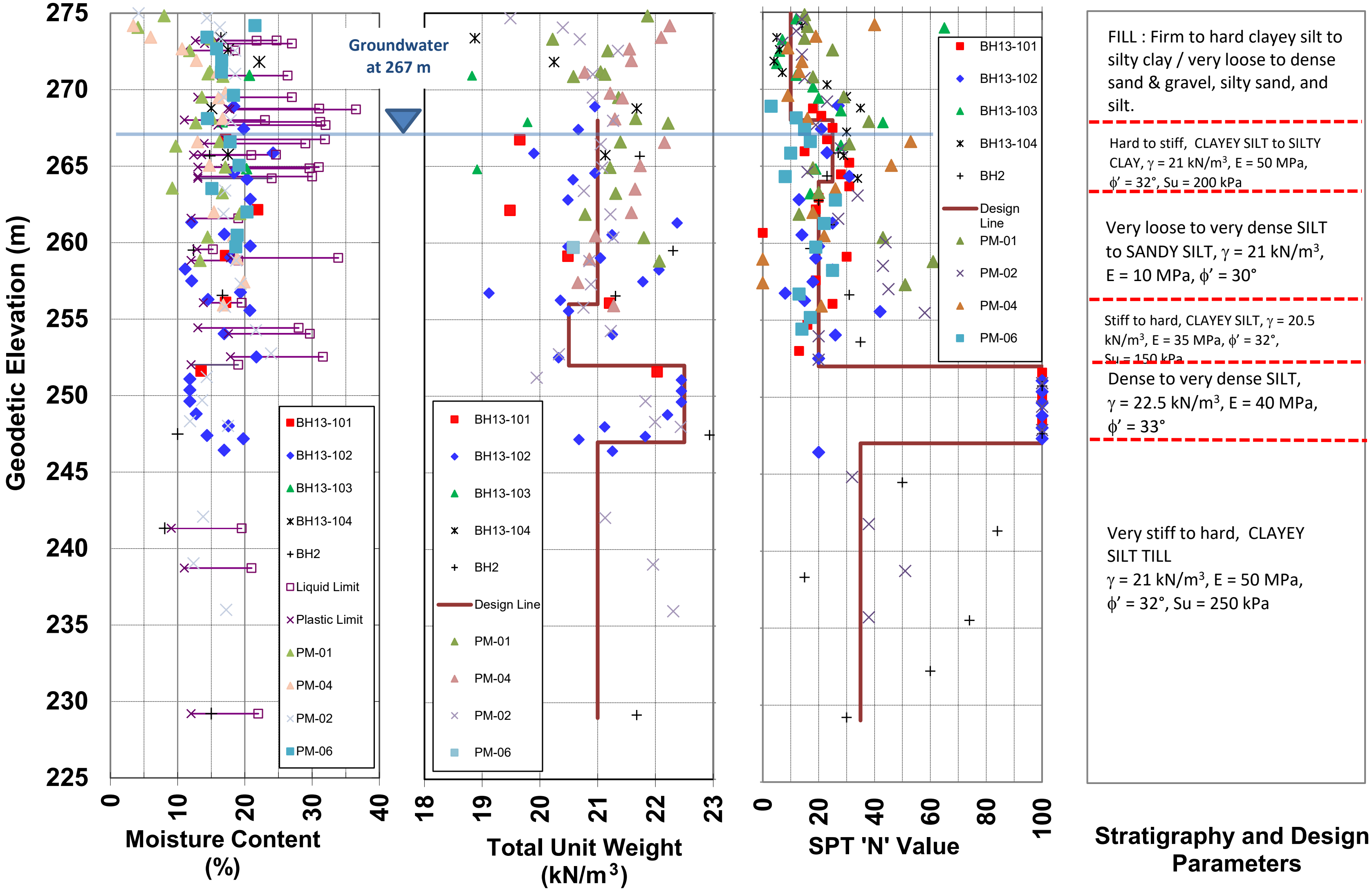
| | | | | | |
|---|---|---|---|---|---|
| Samples Relinquished by (print name & sign): | Date/Time | Samples Received by (Print name & sign): Anthony Dasilva Dasilva | Date/Time | Pink Copy – Client<br>Yellow + Golden Copy AGAT<br>White Copy – AGAT | Page _____ of _____ |
| Samples Relinquished by (print name & sign): | Date/Time | Samples Received by (Print name & sign): | Date/Time | | NO: |

Document ID: DIV-78-1511.006

Date Issued: July 20, 2011

'22 SEP 14 9:56AM

# APPENDIX E



FILL : Firm to hard clayey silt to silty clay / very loose to dense sand & gravel, silty sand, and silt.

Hard to stiff, CLAYEY SILT to SILTY CLAY, $\gamma$ = 21 kN/m$^3$, E = 50 MPa, $\phi'$ = 32°, Su = 200 kPa

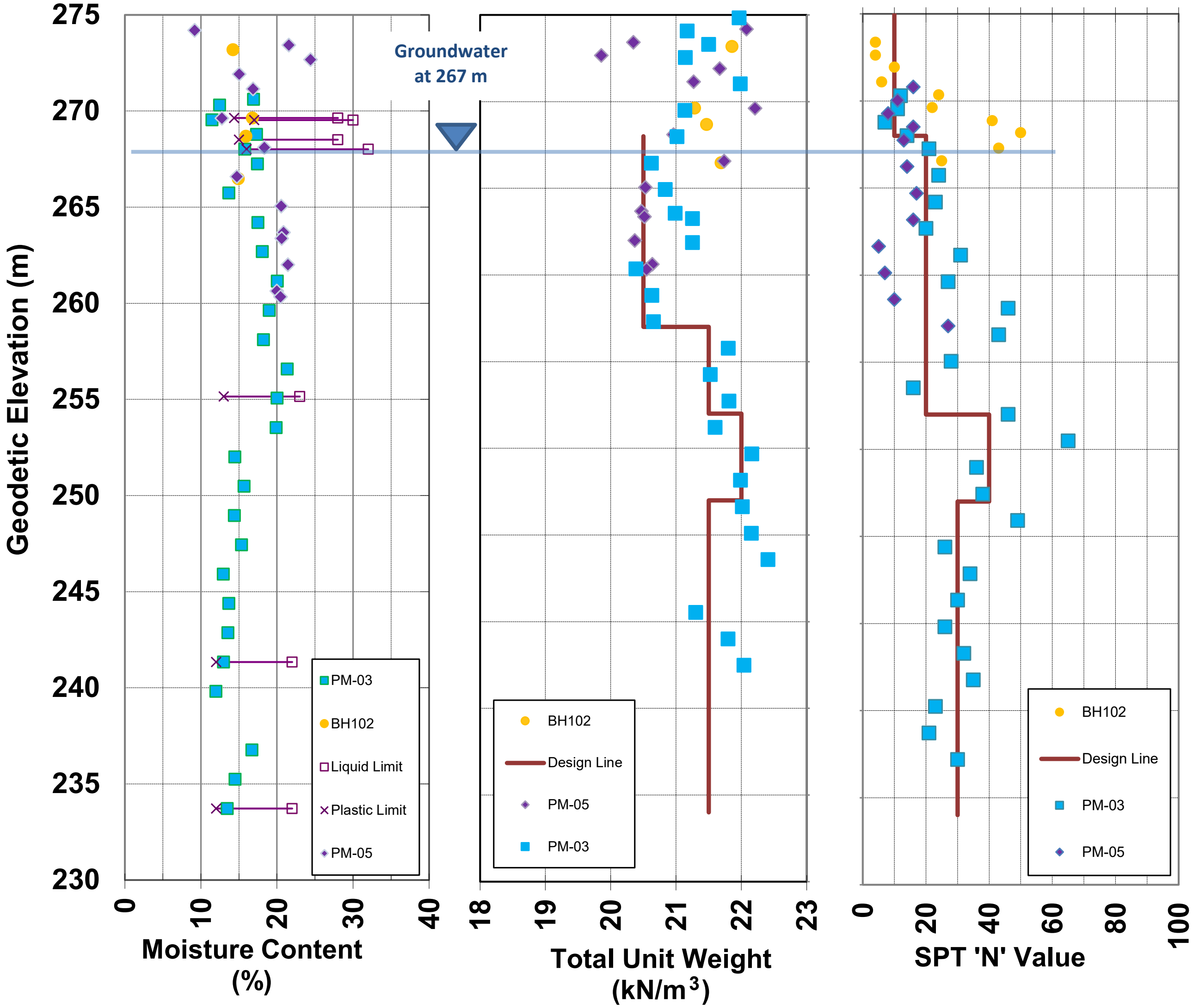
Very loose to very dense SILT to SANDY SILT, $\gamma$ = 21 kN/m$^3$, E = 10 MPa, $\phi'$ = 30°

Stiff to hard, CLAYEY SILT, $\gamma$ = 20.5 kN/m$^3$, E = 35 MPa, $\phi'$ = 32°, Su = 150 kPa

Dense to very dense SILT, $\gamma$ = 22.5 kN/m$^3$, E = 40 MPa, $\phi'$ = 33°

Very stiff to hard, CLAYEY SILT TILL $\gamma$ = 21 kN/m$^3$, E = 50 MPa, $\phi'$ = 32°, Su = 250 kPa

| | | |
|---|---|---|
| Stantec **Stantec Consulting Ltd.** | **Geotechnical Model**<br>**Highway 401 - Pond Mills Road Overpass** | **Figure No. E1** |



FILL : Firm to hard clayey silt to silty clay / very loose to dense sand & gravel, silty sand, and silt.

Hard to stiff, CLAYEY SILT to SILTY CLAY, $\gamma = 20.5$ kN/m$^3$, E = 50 MPa, $\phi' = 32°$, Su = 150 kPa

Loose to dense SILT to SANDY SILT, $\gamma = 20.5$ kN/m$^3$, E = 10 MPa, $\phi' = 30°$

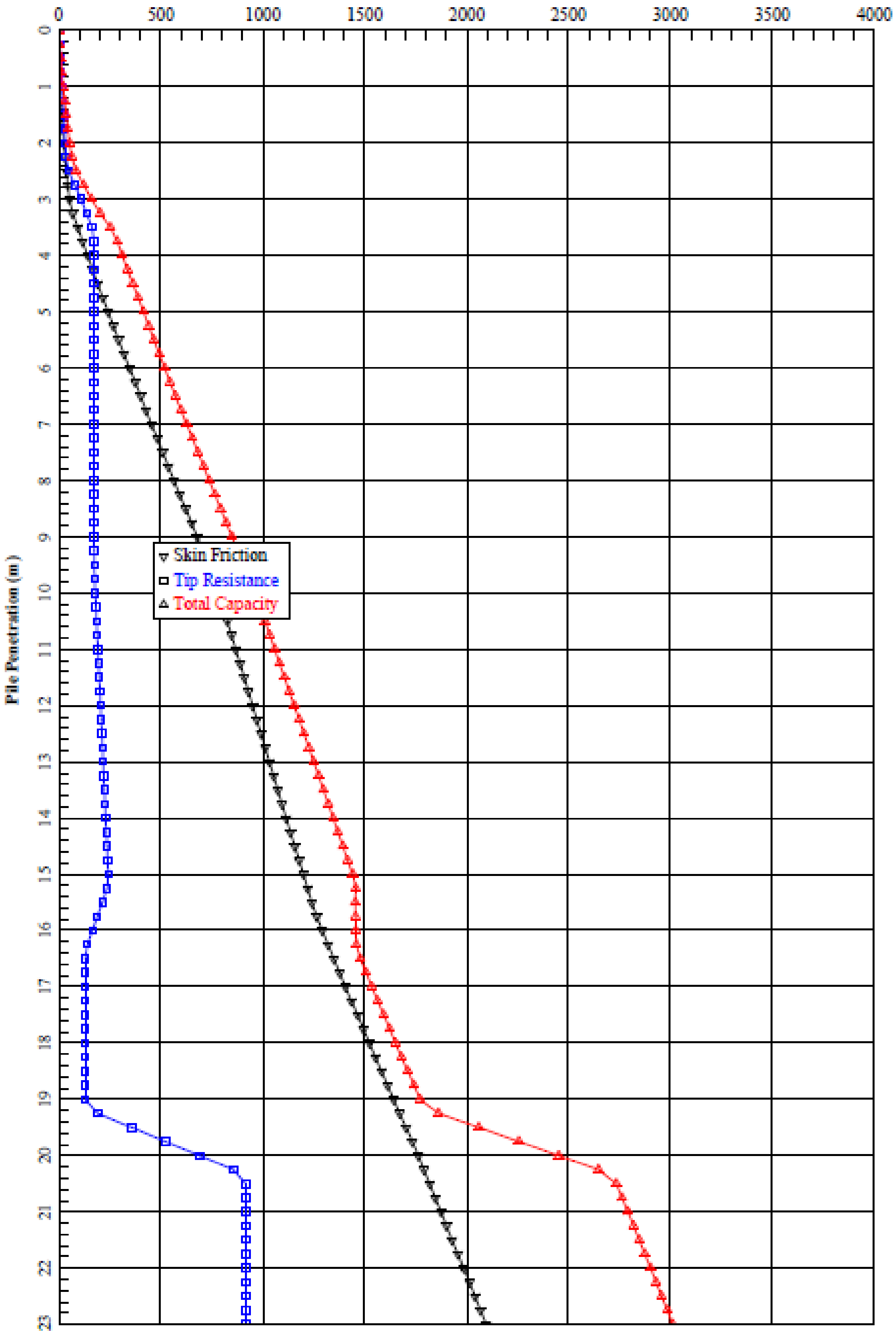
Stiff to hard, SILTY CLAY, $\gamma = 21.5$ kN/m$^3$, E = 35 MPa, $\phi' = 32°$, Su = 150 kPa

Dense to very dense SILT, $\gamma = 22$ kN/m$^3$, E = 40 MPa, $\phi' = 33°$

Very stiff to hard, CLAYEY SILT TILL
$\gamma = 21.5$ kN/m$^3$, E = 50 MPa, $\phi' = 32°$, Su = 200 kPa

**Stratigraphy and Design Parameters**

| | | |
|---|---|---|
| Stantec **Stantec Consulting Ltd.** | **Geotechnical Model**<br>**Highway 401 - Pond Mills Road Overpass**<br>**(the north half of the east abutment)** | **Figure No. E2** |



Stantec

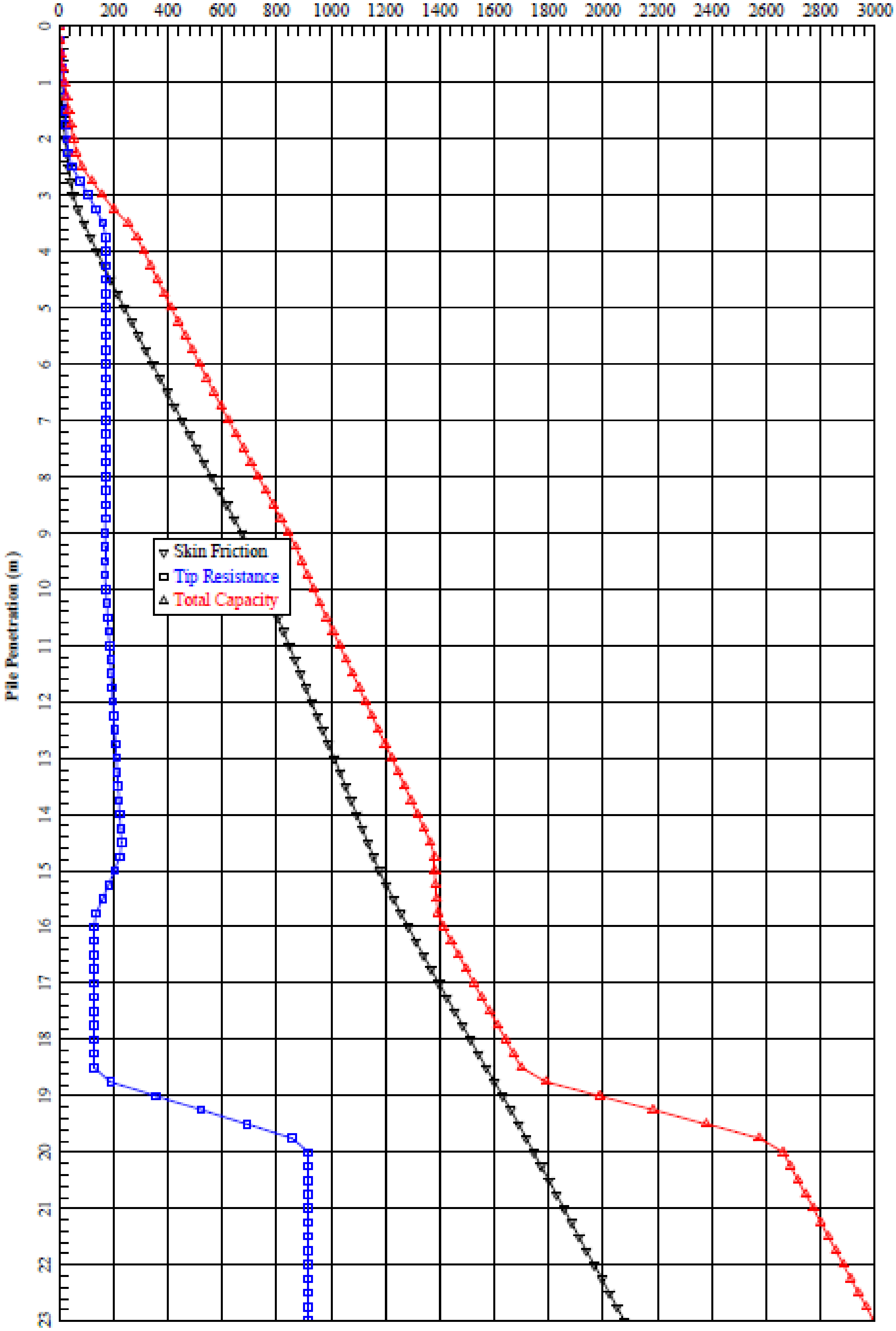
## Unfactored Geotechnical Axial Resistance at ULS in Compression for HP310x110

Pond Mills Road Overpass Replacement (west abutment)

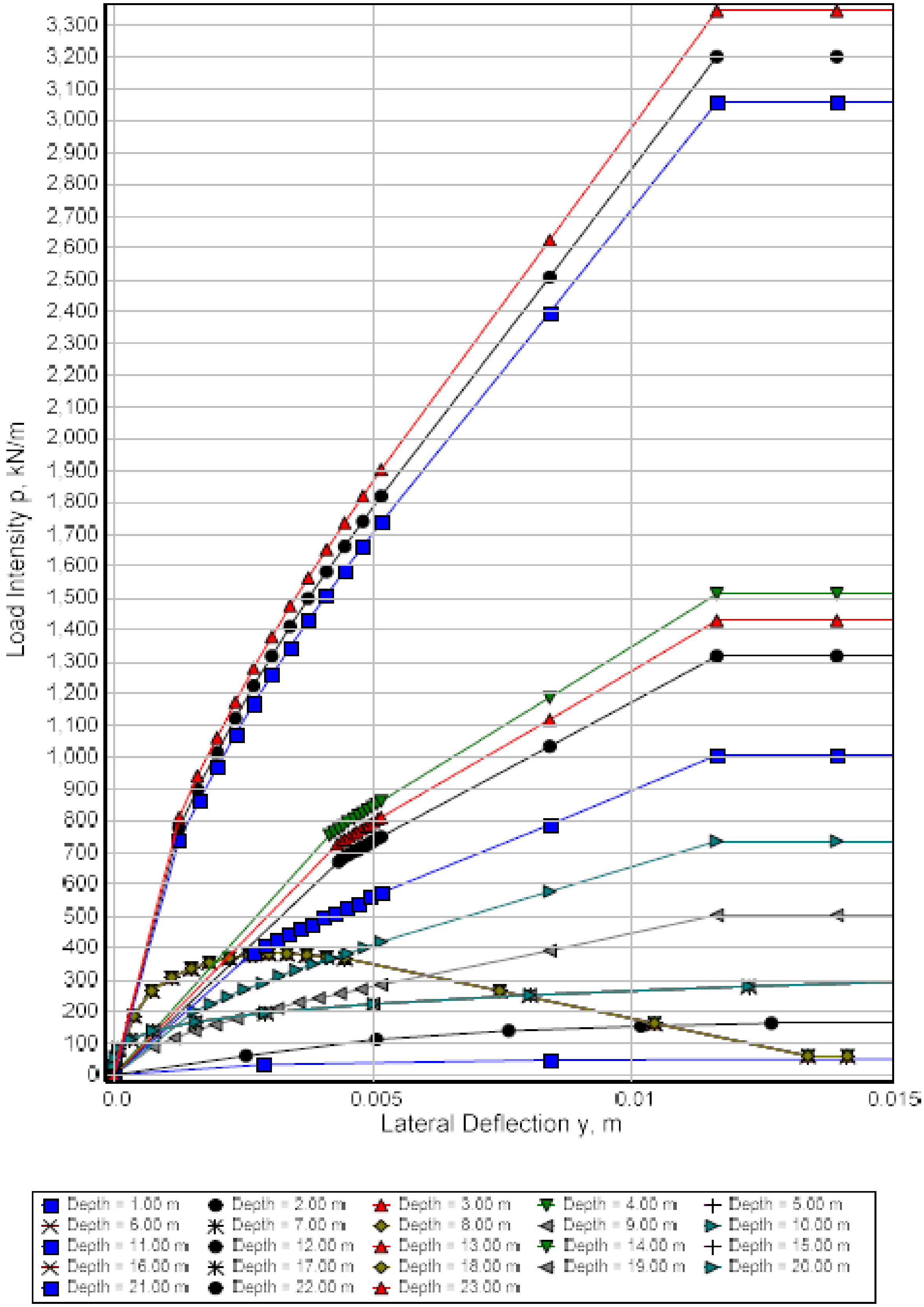
Figure E3

Project No. 165001239

Dufferin Construction Company



| Stantec | Unfactored Geotechnical Axial Resistance at ULS in Compression for HP310x110<br>Pond Mills Road Overpass Replacement (east abutment) | Figure E4<br>Project No. 165001239<br>Dufferin Construction Company |
|---|---|---|



*LPile 2019.11.01, © 2019 by Ensoft, Inc.*

| Stantec | p-y Curves<br>HP310x110 Piles<br>Pond Mills Road Overpass Replacement | Figure E5<br>Project No. 165001239<br>Dufferin Construction Company |
|---|---|---|

**Table E1: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points - HP 310 x 110**

| Depth Below Pile Cap (m) | | Curve Points | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 |
| 1.0 | Y | 0 | 0.00287 | 0.0084 | 0.01903 | 0.03947 | 0.07877 | 0.15434 | 0.29964 | 0.57903 | 1.11626 | 2.14927 | 4.13557 | 7.95491 | 15.2989 | 29.4201 | 56.573 | 108.7836 |
| | P | 0 | 31.4563 | 48.1115 | 49.2862 | 49.2907 | 49.2907 | 49.2907 | 49.2907 | 49.2907 | 49.2907 | 49.2907 | 49.2907 | 49.2907 | 49.2907 | 49.2907 | 49.2907 | 49.29068 |
| 2.0 | Y | 0 | 0.00254 | 0.00508 | 0.00762 | 0.01016 | 0.01269 | 0.01523 | 0.01777 | 0.02031 | 0.02285 | 0.02539 | 0.02793 | 0.03047 | 0.03301 | 0.03554 | 0.03808 | 0.040622 |
| | P | 0 | 62.5558 | 109.782 | 138.417 | 153.534 | 160.934 | 164.422 | 166.037 | 166.779 | 167.118 | 167.273 | 167.343 | 167.376 | 167.39 | 167.397 | 167.4 | 167.4016 |
| 3.0 | Y | 0 | 0.0004 | 0.0007 | 0.00112 | 0.00149 | 0.00186 | 0.00223 | 0.0026 | 0.00298 | 0.00335 | 0.00372 | 0.00409 | 0.00446 | 0.00744 | 0.01042 | 0.01339 | 0.014136 |
| | P | 0 | 186.773 | 264.137 | 307.73 | 336.037 | 355.371 | 368.285 | 376.24 | 380.177 | 380.751 | 378.441 | 373.612 | 366.7 | 264.4 | 162.1 | 59.8 | 59.80001 |
| 4.0 | Y | 0 | 0.00037 | 0.00074 | 0.00112 | 0.00149 | 0.00186 | 0.00223 | 0.0026 | 0.00298 | 0.00335 | 0.00372 | 0.00409 | 0.00446 | 0.00744 | 0.01042 | 0.01339 | 0.014136 |
| | P | 0 | 186.773 | 264.137 | 307.73 | 336.037 | 355.371 | 368.285 | 376.24 | 380.177 | 380.751 | 378.441 | 373.612 | 366.7 | 264.4 | 162.1 | 59.8 | 59.80001 |
| 5.0 | Y | 0 | 0.00037 | 0.00074 | 0.00112 | 0.00149 | 0.00186 | 0.00223 | 0.0026 | 0.00298 | 0.00335 | 0.00372 | 0.00409 | 0.00446 | 0.00744 | 0.01042 | 0.01339 | 0.014136 |
| | P | 0 | 186.773 | 264.137 | 307.73 | 336.037 | 355.371 | 368.285 | 376.24 | 380.177 | 380.751 | 378.441 | 373.612 | 366.7 | 264.4 | 162.1 | 59.8 | 59.80001 |
| 6.0 | Y | 0 | 0.00037 | 0.00074 | 0.00112 | 0.00149 | 0.00186 | 0.00223 | 0.0026 | 0.00298 | 0.00335 | 0.00372 | 0.00409 | 0.00446 | 0.00744 | 0.01042 | 0.01339 | 0.014136 |
| | P | 0 | 186.773 | 264.137 | 307.73 | 336.037 | 355.371 | 368.285 | 376.24 | 380.177 | 380.751 | 378.441 | 373.612 | 366.7 | 264.4 | 162.1 | 59.8 | 59.80001 |
| 7.0 | Y | 0 | 0.00037 | 0.00074 | 0.00112 | 0.00149 | 0.00186 | 0.00223 | 0.0026 | 0.00298 | 0.00335 | 0.00372 | 0.00409 | 0.00446 | 0.00744 | 0.01042 | 0.01339 | 0.014136 |
| | P | 0 | 186.773 | 264.137 | 307.73 | 336.037 | 355.371 | 368.285 | 376.24 | 380.177 | 380.751 | 378.441 | 373.612 | 366.7 | 264.4 | 162.1 | 59.8 | 59.80001 |
| 8.0 | Y | 0 | 0.00037 | 0.00074 | 0.00112 | 0.00149 | 0.00186 | 0.00223 | 0.0026 | 0.00298 | 0.00335 | 0.00372 | 0.00409 | 0.00446 | 0.00744 | 0.01042 | 0.01339 | 0.014136 |
| | P | 0 | 186.773 | 264.137 | 307.73 | 336.037 | 355.371 | 368.285 | 376.24 | 380.177 | 380.751 | 378.441 | 373.612 | 366.7 | 264.4 | 162.1 | 59.8 | 59.80001 |
| 9.0 | Y | 0 | 0.00076 | 0.00116 | 0.00156 | 0.00196 | 0.00236 | 0.00276 | 0.00316 | 0.00357 | 0.00397 | 0.00437 | 0.00477 | 0.00517 | 0.0084 | 0.01163 | 0.01395 | 0.016275 |
| | P | 0 | 88.8108 | 114.802 | 137.434 | 157.876 | 176.735 | 194.376 | 211.039 | 226.893 | 242.062 | 256.642 | 270.705 | 284.313 | 392.352 | 500.391 | 500.391 | 500.3908 |
| 10.0 | Y | 0 | 0.00154 | 0.00187 | 0.0022 | 0.00253 | 0.00286 | 0.00319 | 0.00352 | 0.00385 | 0.00418 | 0.00451 | 0.00484 | 0.00517 | 0.0084 | 0.01163 | 0.01395 | 0.016275 |
| | P | 0 | 199.577 | 224.526 | 247.801 | 269.743 | 290.59 | 310.513 | 329.643 | 348.083 | 365.914 | 383.2 | 399.998 | 416.353 | 574.567 | 732.781 | 732.781 | 732.7812 |
| 11.0 | Y | 0 | 0.00271 | 0.00293 | 0.00316 | 0.00338 | 0.0036 | 0.00383 | 0.00405 | 0.00427 | 0.0045 | 0.00472 | 0.00494 | 0.00517 | 0.0084 | 0.01163 | 0.01395 | 0.016275 |
| | P | 0 | 385.794 | 404.837 | 423.32 | 441.297 | 458.813 | 475.908 | 492.616 | 508.966 | 524.984 | 540.694 | 556.114 | 571.263 | 788.344 | 1005.42 | 1005.42 | 1005.424 |
| 12.0 | Y | 0 | 0.00433 | 0.0044 | 0.00448 | 0.00456 | 0.00463 | 0.00471 | 0.00478 | 0.00486 | 0.00494 | 0.00501 | 0.00509 | 0.00517 | 0.0084 | 0.01163 | 0.01395 | 0.016275 |
| | P | 0 | 672.026 | 679.222 | 686.37 | 693.47 | 700.524 | 707.532 | 714.495 | 721.415 | 728.293 | 735.129 | 741.924 | 748.68 | 1033.18 | 1317.68 | 1317.68 | 1317.676 |
| 13.0 | Y | 0 | 0.00431 | 0.00439 | 0.00447 | 0.00454 | 0.00462 | 0.0047 | 0.00478 | 0.00486 | 0.00493 | 0.00501 | 0.00509 | 0.00517 | 0.0084 | 0.01163 | 0.01395 | 0.016275 |
| | P | 0 | 725.602 | 733.531 | 741.406 | 749.227 | 756.995 | 764.713 | 772.381 | 780 | 787.571 | 795.096 | 802.575 | 810.009 | 1117.81 | 1425.62 | 1425.62 | 1425.616 |
| 14.0 | Y | 0 | 0.00416 | 0.00425 | 0.00434 | 0.00443 | 0.00452 | 0.00462 | 0.00471 | 0.0048 | 0.00489 | 0.00498 | 0.00507 | 0.00517 | 0.0084 | 0.01163 | 0.01395 | 0.016275 |
| | P | 0 | 753.377 | 763.453 | 773.444 | 783.352 | 793.18 | 802.931 | 812.605 | 822.206 | 831.735 | 841.195 | 850.586 | 859.91 | 1186.68 | 1513.44 | 1513.44 | 1513.443 |
| 15.0 | Y | 0 | 1.2E-06 | 2E-05 | 9.9E-05 | 0.00031 | 0.00077 | 0.00159 | 0.00294 | 0.00502 | 0.00804 | 0.01225 | 0.01793 | 0.0254 | 0.03498 | 0.04705 | 0.062 | 0.0775 |
| | P | 0 | 27.9 | 55.8 | 83.7 | 111.6 | 139.5 | 167.4 | 195.3 | 223.2 | 251.1 | 279 | 306.9 | 334.8 | 362.7 | 390.6 | 418.5 | 418.5 |
| 16.0 | Y | 0 | 1.2E-06 | 2E-05 | 9.9E-05 | 0.00031 | 0.00077 | 0.00159 | 0.00294 | 0.00502 | 0.00804 | 0.01225 | 0.01793 | 0.0254 | 0.03498 | 0.04705 | 0.062 | 0.0775 |
| | P | 0 | 27.9 | 55.8 | 83.7 | 111.6 | 139.5 | 167.4 | 195.3 | 223.2 | 251.1 | 279 | 306.9 | 334.8 | 362.7 | 390.6 | 418.5 | 418.5 |
| 17.0 | Y | 0 | 1.2E-06 | 2E-05 | 9.9E-05 | 0.00031 | 0.00077 | 0.00159 | 0.00294 | 0.00502 | 0.00804 | 0.01225 | 0.01793 | 0.0254 | 0.03498 | 0.04705 | 0.062 | 0.0775 |
| | P | 0 | 27.9 | 55.8 | 83.7 | 111.6 | 139.5 | 167.4 | 195.3 | 223.2 | 251.1 | 279 | 306.9 | 334.8 | 362.7 | 390.6 | 418.5 | 418.5 |

| | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 18.0 | Y | 0 | 1.2E-06 | 2E-05 | 9.9E-05 | 0.00031 | 0.00077 | 0.00159 | 0.00294 | 0.00502 | 0.00804 | 0.01225 | 0.01793 | 0.0254 | 0.03498 | 0.04705 | 0.062 | 0.0775 |
| | P | 0 | 27.9 | 55.8 | 83.7 | 111.6 | 139.5 | 167.4 | 195.3 | 223.2 | 251.1 | 279 | 306.9 | 334.8 | 362.7 | 390.6 | 418.5 | 418.5 |
| 19.0 | Y | 0 | 1.2E-06 | 2E-05 | 9.9E-05 | 0.00031 | 0.00077 | 0.00159 | 0.00294 | 0.00502 | 0.00804 | 0.01225 | 0.01793 | 0.0254 | 0.03498 | 0.04705 | 0.062 | 0.0775 |
| | P | 0 | 27.9 | 55.8 | 83.7 | 111.6 | 139.5 | 167.4 | 195.3 | 223.2 | 251.1 | 279 | 306.9 | 334.8 | 362.7 | 390.6 | 418.5 | 418.5 |
| 20.0 | Y | 0 | 1.2E-06 | 2E-05 | 9.9E-05 | 0.00031 | 0.00077 | 0.00159 | 0.00294 | 0.00502 | 0.00804 | 0.01225 | 0.01793 | 0.0254 | 0.03498 | 0.04705 | 0.062 | 0.0775 |
| | P | 0 | 27.9 | 55.8 | 83.7 | 111.6 | 139.5 | 167.4 | 195.3 | 223.2 | 251.1 | 279 | 306.9 | 334.8 | 362.7 | 390.6 | 418.5 | 418.5 |
| 21.0 | Y | 0 | 0.00127 | 0.00163 | 0.00198 | 0.00233 | 0.00269 | 0.00304 | 0.0034 | 0.00375 | 0.0041 | 0.00446 | 0.00481 | 0.00517 | 0.0084 | 0.01163 | 0.01395 | 0.016275 |
| | P | 0 | 740.372 | 859.572 | 968.93 | 1070.84 | 1166.84 | 1258 | 1345.08 | 1428.66 | 1509.21 | 1587.07 | 1662.54 | 1735.86 | 2395.49 | 3055.12 | 3055.12 | 3055.122 |
| 22.0 | Y | 0 | 0.00127 | 0.00163 | 0.00198 | 0.00233 | 0.00269 | 0.00304 | 0.0034 | 0.00375 | 0.0041 | 0.00446 | 0.00481 | 0.00517 | 0.0084 | 0.01163 | 0.01395 | 0.016275 |
| | P | 0 | 775.058 | 899.958 | 1014.54 | 1121.31 | 1221.89 | 1317.39 | 1408.62 | 1496.18 | 1580.56 | 1662.12 | 1741.19 | 1818 | 2508.84 | 3199.68 | 3199.68 | 3199.682 |
| 23.0 | Y | 0 | 0.00127 | 0.00162 | 0.00198 | 0.00233 | 0.00269 | 0.00304 | 0.0034 | 0.00375 | 0.0041 | 0.00446 | 0.00481 | 0.00517 | 0.0084 | 0.01163 | 0.01395 | 0.016275 |
| | P | 0 | 809.745 | 940.345 | 1060.15 | 1171.78 | 1276.94 | 1376.78 | 1472.15 | 1563.7 | 1651.91 | 1737.18 | 1819.84 | 1900.14 | 2622.19 | 3344.24 | 3344.24 | 3344.243 |

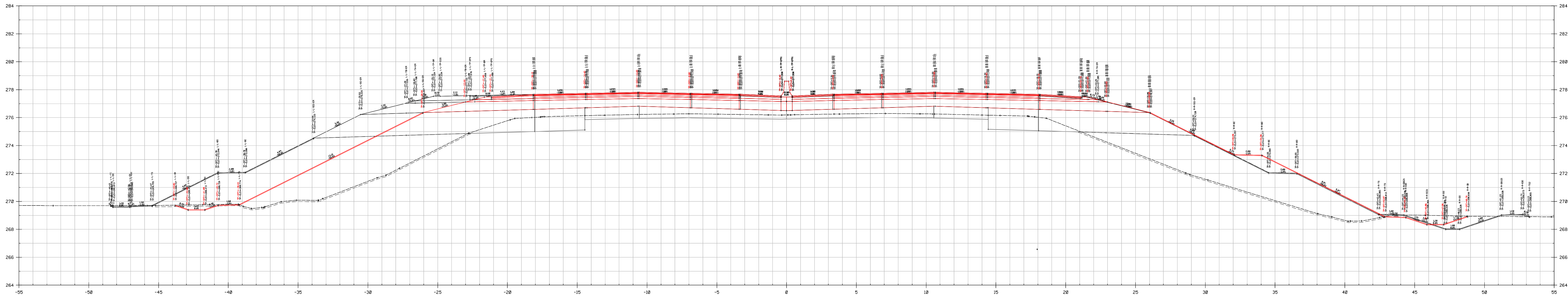
The response of a pile to lateral loads is a nonlinear relationship. The p-y geotechnical approach was used to estimate the anticipated deformation of a pile within the soil medium. The p-y curves represent the load-deformation characteristics of elastic-plastic springs with a non-linear response within the elastic range. These non-linear elastic-plastic springs provide a more realistic representation or modeling of the soil pressure response against the face of the pile. The table presents the Load Intensity per unit length of pile p (kN/m) vs Lateral Deflection y (m). The p-y points can be used for the structural design of the pile in response to lateral loads. Where spring spacings of less than 1.0 m are proposed, the tabulated “p” values are to be multiplied by the actual spring spacing; i.e. by 0.25 for 0.25 m spacings.

# APPENDIX F

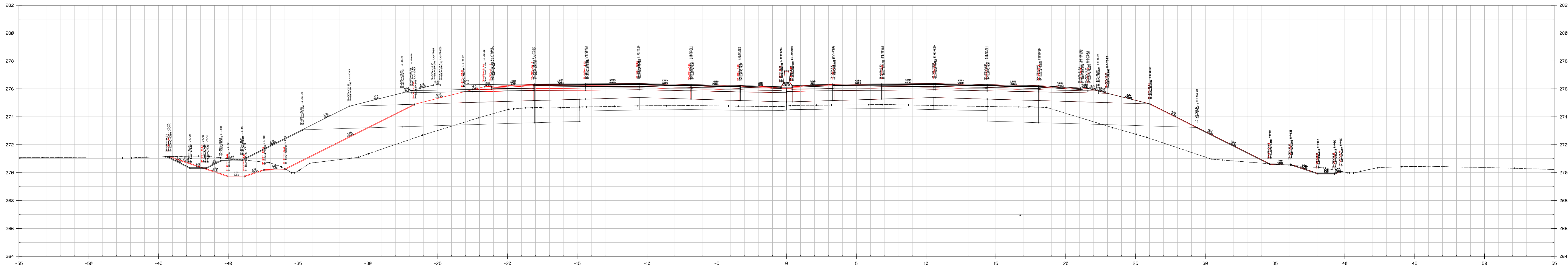
## F.1 SELECTED HIGHWAY EMBANKMENT CROSS SECTIONS

## F.2 SETTLEMENT ANALYSIS RESULTS

## F.3 SLOPE STABILITY ANALYSIS RESULTS



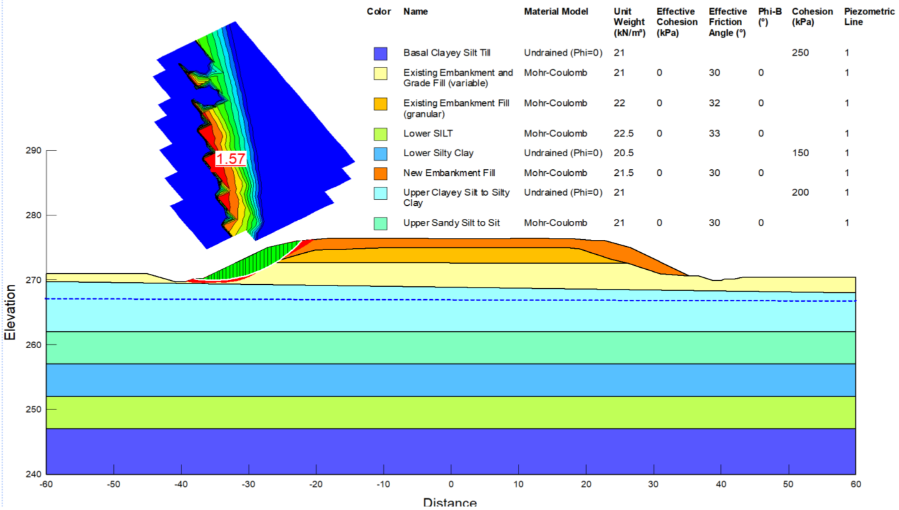
25+550.00



25+675.00

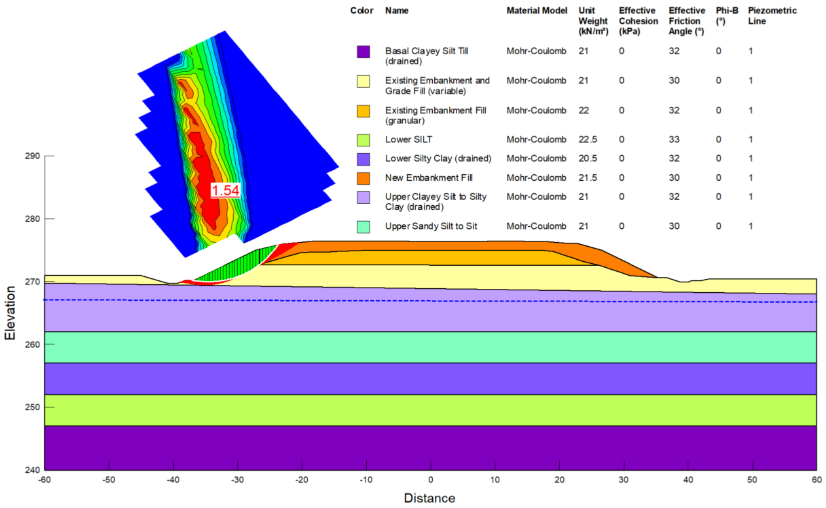
C:\CAD Drawings\Acad2019 Drawings\2023\165001239\Cross Section Comparison\1239_Highbury_cross_section_comparison.dwg Printed: Jan 25, 2023

Undrained Condition



| Color | Name | Material Model | Unit Weight (kN/m³) | Effective Cohesion (kPa) | Effective Friction Angle (°) | Phi-B (°) | Cohesion (kPa) | Piezometric Line |
|---|---|---|---|---|---|---|---|---|
| 🟦 | Basal Clayey Silt Till | Undrained (Phi=0) | 21 | | | | 250 | 1 |
| 🟨 | Existing Embankment and Grade Fill (variable) | Mohr-Coulomb | 21 | 0 | 30 | 0 | | 1 |
| 🟧 | Existing Embankment Fill (granular) | Mohr-Coulomb | 22 | 0 | 32 | 0 | | 1 |
| 🟩 | Lower SILT | Mohr-Coulomb | 22.5 | 0 | 33 | 0 | | 1 |
| 🟦 | Lower Silty Clay | Undrained (Phi=0) | 20.5 | | | | 150 | 1 |
| 🟧 | New Embankment Fill | Mohr-Coulomb | 21.5 | 0 | 30 | 0 | | 1 |
| 🟦 | Upper Clayey Silt to Silty Clay | Undrained (Phi=0) | 21 | | | | 200 | 1 |
| 🟩 | Upper Sandy Silt to Sit | Mohr-Coulomb | 21 | 0 | 30 | 0 | | 1 |

Drained Condition



| Color | Name | Material Model | Unit Weight (kN/m³) | Effective Cohesion (kPa) | Effective Friction Angle (°) | Phi-B (°) | Piezometric Line |
|---|---|---|---|---|---|---|---|
| 🟪 | Basal Clayey Silt Till (drained) | Mohr-Coulomb | 21 | 0 | 32 | 0 | 1 |
| 🟨 | Existing Embankment and Grade Fill (variable) | Mohr-Coulomb | 21 | 0 | 30 | 0 | 1 |
| 🟧 | Existing Embankment Fill (granular) | Mohr-Coulomb | 22 | 0 | 32 | 0 | 1 |
| 🟩 | Lower SILT | Mohr-Coulomb | 22.5 | 0 | 33 | 0 | 1 |
| 🟪 | Lower Silty Clay (drained) | Mohr-Coulomb | 20.5 | 0 | 32 | 0 | 1 |
| 🟧 | New Embankment Fill | Mohr-Coulomb | 21.5 | 0 | 30 | 0 | 1 |
| 🟪 | Upper Clayey Silt to Silty Clay (drained) | Mohr-Coulomb | 21 | 0 | 32 | 0 | 1 |
| 🟩 | Upper Sandy Silt to Sit | Mohr-Coulomb | 21 | 0 | 30 | 0 | 1 |

Stantec

# Slope Stability Analysis (Static)

Deep Seated Failure

STA 25+675 (Pond Mills Road, LT Side)

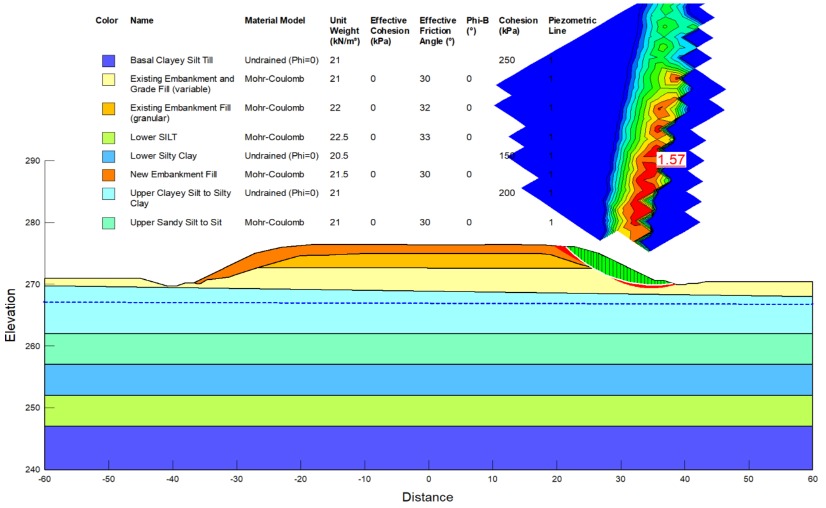
Pond Mills Road Overpass Replacement

Figure F1

Project No. 165001239

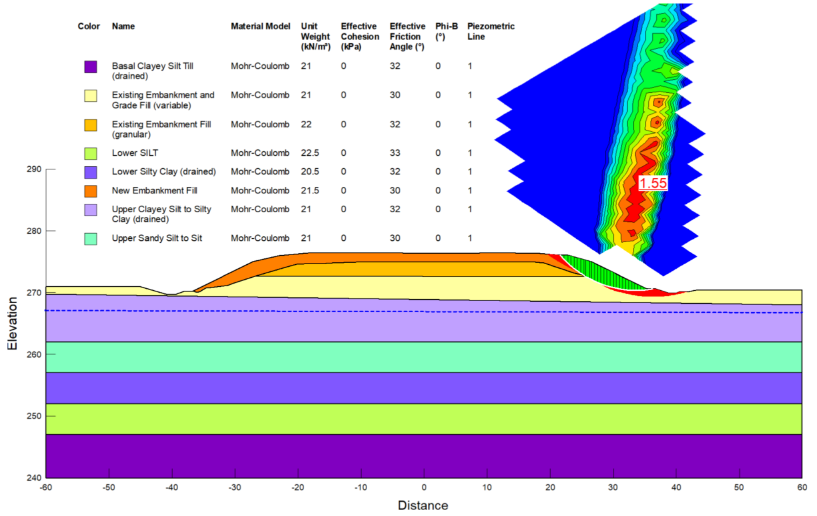
GWP No. 3032-11-00

Undrained
Condition



| Color | Name | Material Model | Unit Weight (kN/m³) | Effective Cohesion (kPa) | Effective Friction Angle (°) | Phi-B (°) | Cohesion (kPa) | Piezometric Line |
|---|---|---|---|---|---|---|---|---|
| 🟦 | Basal Clayey Silt Till | Undrained (Phi=0) | 21 | | | | 250 | 1 |
| 🟨 | Existing Embankment and Grade Fill (variable) | Mohr-Coulomb | 21 | 0 | 30 | 0 | | 1 |
| 🟧 | Existing Embankment Fill (granular) | Mohr-Coulomb | 22 | 0 | 32 | 0 | | 1 |
| 🟩 | Lower SILT | Mohr-Coulomb | 22.5 | 0 | 33 | 0 | | 1 |
| 🟦 | Lower Silty Clay | Undrained (Phi=0) | 20.5 | | | | 150 | 1 |
| 🟧 | New Embankment Fill | Mohr-Coulomb | 21.5 | 0 | 30 | 0 | | 1 |
| 🟦 | Upper Clayey Silt to Silty Clay | Undrained (Phi=0) | 21 | | | | 200 | 1 |
| 🟩 | Upper Sandy Silt to Sit | Mohr-Coulomb | 21 | 0 | 30 | 0 | | 1 |

Drained
Condition



| Color | Name | Material Model | Unit Weight (kN/m³) | Effective Cohesion (kPa) | Effective Friction Angle (°) | Phi-B (°) | Piezometric Line |
|---|---|---|---|---|---|---|---|
| 🟪 | Basal Clayey Silt Till (drained) | Mohr-Coulomb | 21 | 0 | 32 | 0 | 1 |
| 🟨 | Existing Embankment and Grade Fill (variable) | Mohr-Coulomb | 21 | 0 | 30 | 0 | 1 |
| 🟧 | Existing Embankment Fill (granular) | Mohr-Coulomb | 22 | 0 | 32 | 0 | 1 |
| 🟩 | Lower SILT | Mohr-Coulomb | 22.5 | 0 | 33 | 0 | 1 |
| 🟪 | Lower Silty Clay (drained) | Mohr-Coulomb | 20.5 | 0 | 32 | 0 | 1 |
| 🟧 | New Embankment Fill | Mohr-Coulomb | 21.5 | 0 | 30 | 0 | 1 |
| 🟪 | Upper Clayey Silt to Silty Clay (drained) | Mohr-Coulomb | 21 | 0 | 32 | 0 | 1 |
| 🟩 | Upper Sandy Silt to Sit | Mohr-Coulomb | 21 | 0 | 30 | 0 | 1 |

Stantec

# Slope Stability Analysis (Static)

Deep Seated Failure

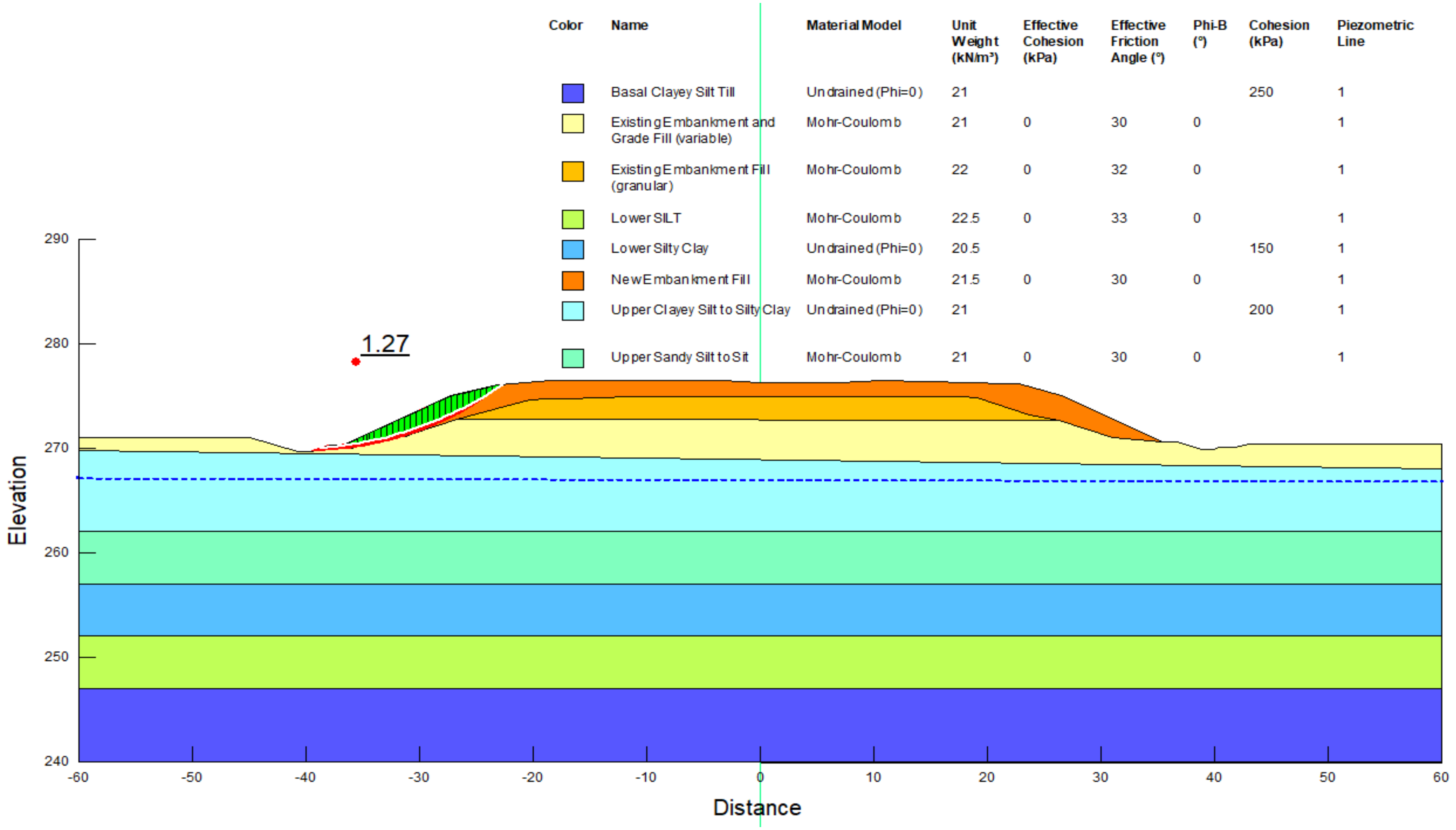
STA 25+675 (Pond Mills Road, RT Side)

Pond Mills Road Overpass Replacement

Figure F2

Project No. 165001239

GWP No. 3032-11-00



| Color | Name | Material Model | Unit Weight (kN/m³) | Effective Cohesion (kPa) | Effective Friction Angle (°) | Phi-B (°) | Cohesion (kPa) | Piezometric Line |
|---|---|---|---|---|---|---|---|---|
| 🟦 | Basal Clayey Silt Till | Undrained (Phi=0) | 21 | | | | 250 | 1 |
| 🟨 | Existing Embankment and Grade Fill (variable) | Mohr-Coulomb | 21 | 0 | 30 | 0 | | 1 |
| 🟧 | Existing Embankment Fill (granular) | Mohr-Coulomb | 22 | 0 | 32 | 0 | | 1 |
| 🟩 | Lower SILT | Mohr-Coulomb | 22.5 | 0 | 33 | 0 | | 1 |
| 🟦 | Lower Silty Clay | Undrained (Phi=0) | 20.5 | | | | 150 | 1 |
| 🟧 | New Embankment Fill | Mohr-Coulomb | 21.5 | 0 | 30 | 0 | | 1 |
| 🟦 | Upper Clayey Silt to Silty Clay | Undrained (Phi=0) | 21 | | | | 200 | 1 |
| 🟩 | Upper Sandy Silt to Silt | Mohr-Coulomb | 21 | 0 | 30 | 0 | | 1 |

Stantec

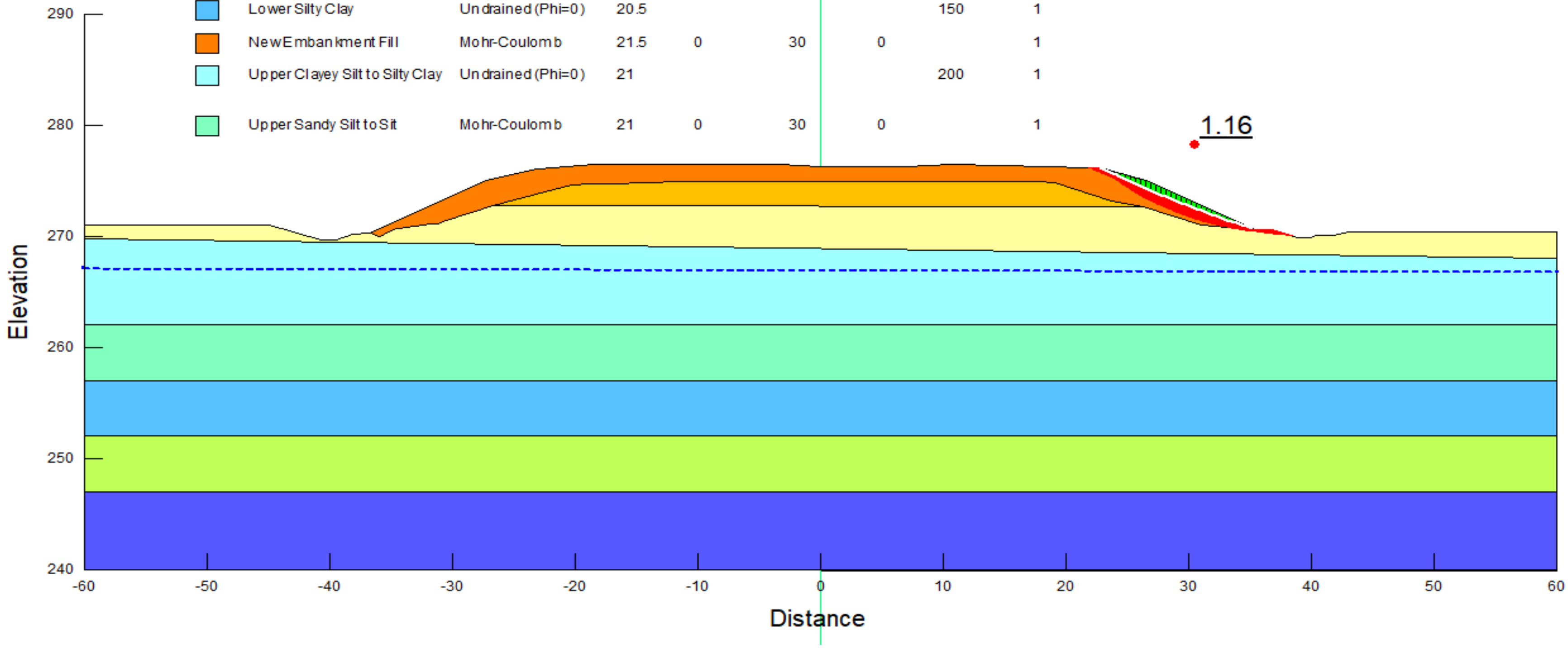
# Pseudo-Static Slope Stability Analysis
STA 25+675 (Pond Mills Road, LT Side)
Pond Mills Road Overpass Replacement

Figure F3

Project No. 165001239

GWP No. 3032-11-00

| Color | Name | Material Model | Unit Weight (kN/m³) | Effective Cohesion (kPa) | Effective Friction Angle (°) | Phi-B (°) | Cohesion (kPa) | Piezometric Line |
|---|---|---|---|---|---|---|---|---|
| 🟦 | Basal Clayey Silt Till | Undrained (Phi=0) | 21 | | | | 250 | 1 |
| 🟨 | Existing Embankment and Grade Fill (variable) | Mohr-Coulomb | 21 | 0 | 30 | 0 | | 1 |
| 🟧 | Existing Embankment Fill (granular) | Mohr-Coulomb | 22 | 0 | 32 | 0 | | 1 |
| 🟩 | Lower SILT | Mohr-Coulomb | 22.5 | 0 | 33 | 0 | | 1 |
| 🟦 | Lower Silty Clay | Undrained (Phi=0) | 20.5 | | | | 150 | 1 |
| 🟧 | New Embankment Fill | Mohr-Coulomb | 21.5 | 0 | 30 | 0 | | 1 |
| 🟦 | Upper Clayey Silt to Silty Clay | Undrained (Phi=0) | 21 | | | | 200 | 1 |
| 🟩 | Upper Sandy Silt to Silt | Mohr-Coulomb | 21 | 0 | 30 | 0 | | 1 |



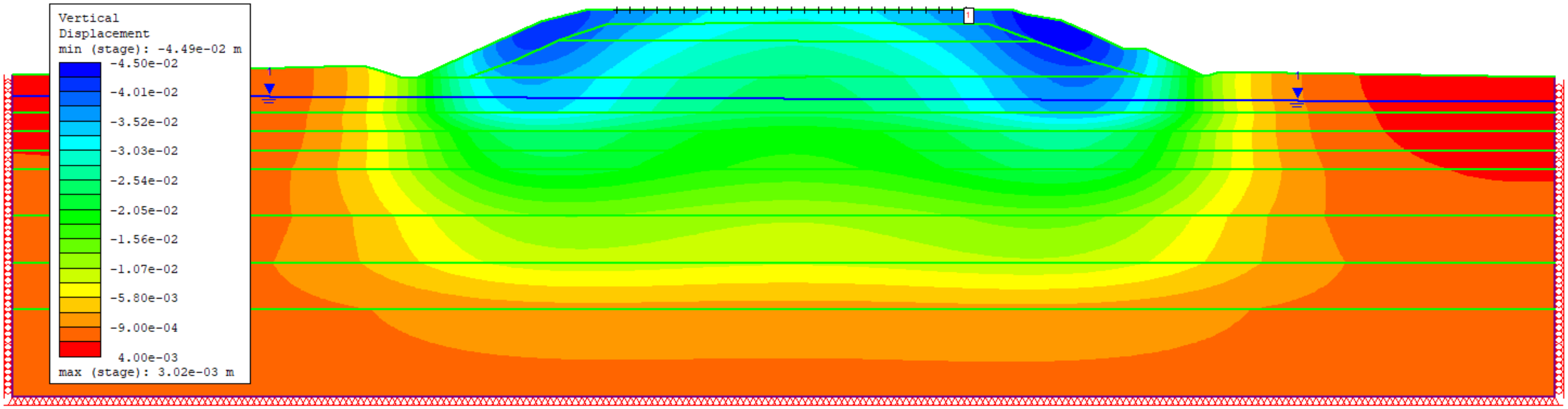
Stantec

# Pseudo-Static Slope Stability Analysis
STA 25+675 (Pond Mills Road, RT Side)
Pond Mills Road Overpass Replacement

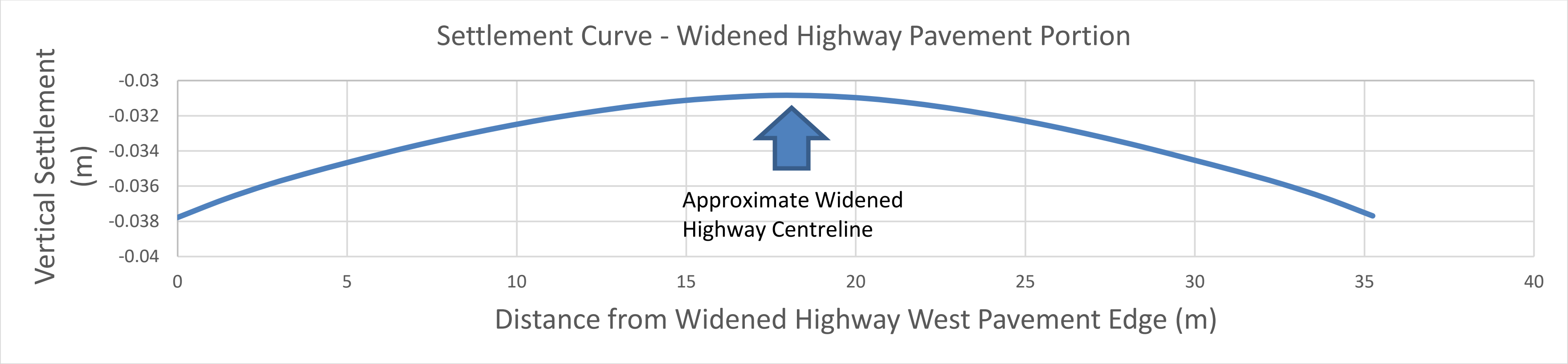
Figure F4

Project No. 165001239

GWP No. 3032-11-00



Vertical Displacement (m)



Stantec

# Embankment Settlement

## 2D Finite Element Analysis

Embankment Close to the Pond Mills Road Bridge (Station 25+550)

Pond Mills Road Overpass Replacement

Figure F5

Project No. 165001239

GWP No. 3032-11-00

# APPENDIX G

## G.1 2015 NATIONAL BUILDING CODE SEISMIC HAZARD CALCULATIONS

# 2015 National Building Code Seismic Hazard Calculation

**INFORMATION:** **Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836**
**Western Canada English (250) 363-6500 Facsimile (250) 363-6565**

**Site:** 42.934N 81.192W **User File Reference:** Pond Mills Road 2022-10-30 08:34 UT

**Requested by:** Gwangha Roh, Stantec

| Probability of exceedance per annum | 0.000404 | 0.001 | 0.0021 | 0.01 |
|---|---|---|---|---|
| Probability of exceedance in 50 years | 2 % | 5 % | 10 % | 40 % |
| Sa (0.05) | 0.089 | 0.051 | 0.031 | 0.009 |
| Sa (0.1) | 0.119 | 0.071 | 0.045 | 0.014 |
| Sa (0.2) | 0.111 | 0.068 | 0.044 | 0.015 |
| Sa (0.3) | 0.091 | 0.057 | 0.038 | 0.014 |
| Sa (0.5) | 0.071 | 0.045 | 0.030 | 0.011 |
| Sa (1.0) | 0.041 | 0.027 | 0.018 | 0.005 |
| Sa (2.0) | 0.021 | 0.013 | 0.008 | 0.002 |
| Sa (5.0) | 0.005 | 0.003 | 0.002 | 0.001 |
| Sa (10.0) | 0.002 | 0.001 | 0.001 | 0.000 |
| PGA (g) | 0.067 | 0.040 | 0.025 | 0.008 |
| PGV (m/s) | 0.056 | 0.034 | 0.021 | 0.006 |

**Notes:** Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s$^2$). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

---

## References

**National Building Code of Canada 2015 NRCC no. 56190;** Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

**Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)**
**Commentary J**: Design for Seismic Effects

**Geological Survey of Canada Open File 7893** Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information