THURBER ENGINEERING LTD.

# FOUNDATION INVESTIGATION AND DESIGN REPORT
# FREDERICK STREET UNDERPASS
# HIGHWAY 7- NEW, KITCHENER TO GUELPH
# G.W.P. 408-88-00

**GEOCRES NO. 40P8-285**

**Latitude 43.458853°, Longitude -80.471394°**

**Report**

**to**

**WSP**

Date: February 9, 2021
File: 11375

103, 2010 Winston Park Drive, Oakville, ON L6H 5R7 T: 905 829 8666 F: 905 829 1166
thurber.ca

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**PART 1: FACTUAL INFORMATION**

## 1 INTRODUCTION

This report presents the factual findings obtained from a detailed foundation investigation conducted at the site of the proposed structure that will carry the eastbound lanes (EBL) and westbound lanes (WBL) of Frederick Street over the Kitchener-Waterloo Expressway (KWE) in the Regional Municipality of Waterloo, Ontario.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, a stratigraphic profile, cross sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions under the potential foundation footprint was developed from the data obtained in the course of the investigation.

Thurber was retained by WSP to carry out the site investigation under the Ministry of Transportation Ontario (MTO) Agreement Order Number 3014-E-0013.

Reference has been made to information on subsurface conditions contained in previous foundation reports for the site. The titles of the reports are listed as follows:

- Foundation Investigation Report for Frederick Street Underpass, Kitchener-Waterloo Expressway, District #4 (Hamilton), W.J. 66-F-53, W.P. 634-64, Geocres No. 40-P8-48, prepared by DHO (Department of Highways Ontario), dated July 21, 1966, (Reference 1)

- Preliminary Foundation Investigation and Design Report for Frederick Street Underpass, Highway 7-New, Kitchener to Guelph, G.W.P. 408-88-00, Geocres No. 40P8-203, prepared by Thurber Engineering Ltd, dated November 13, 2012 (Reference 2)

- Foundation investigation and design report for Northeast Corner Retaining Wall, Frederick Street Underpass, Site No. 33-234, G.W.P. 3110-09-00, City of Kitchener, Ontario, Geocres No. 40P8-199, prepared by Peto MacCallum Ltd, dated May 31, 2012 (Reference 3)

## 2 SITE DESCRIPTION

The site is located in the City of Kitchener, approximately 350 m south of the KWE and Victoria Street interchange. At this location, an underpass structure carries Frederick Street over the northbound and southbound lanes (NBL and SBL) of the KWE and existing ramps (E-S and S-E). The existing underpass at KWE and Frederick Street is a four-span structure supported on two abutments and three piers. The original 1959 GA drawing for the structure indicates that the existing abutments and piers are supported on spread footings.

The existing grade on Frederick Street is at about Elev. 327.5 m and 325.0 m adjacent to the west and east abutments, respectively. The KWE has been constructed in a cut up to about 6.5 m deep and the existing KWE grade ranges from about Elev. 321 to 320 m, decreasing towards the east. The site is primarily surrounded by industrial and commercial lands and is relatively flat.

Based on the Ontario Geological Survey Special Volume 2, The Physiography of Southern Ontario, Third Edition by Chapman and Putnam, the site lies within the physiographic region known as the Waterloo Hills, characterized by ridges of sandy till and kames or kame moraines, with outwash sands occupying the intervening hollows.

The following photographs of the site are included in Appendix F:

1. A photograph of the existing east abutment

2. A photograph of the existing west abutment

## 3 SITE INVESTIGATION AND FIELD TESTING

A previous investigation conducted in 1966 at this site (Reference 1) consisted of drilling and sampling a total of nine boreholes (numbered 2, 3, 6, 7, 10, 11, 14, 16 and 17) and sixteen dynamic cone penetration tests (DCPTs). Nine DCPTs were conducted adjacent to the boreholes and seven DCPTS were conducted at various locations within the underpass area.

The current investigation was completed in August 2020 and involved the completion of two boreholes numbered BH20-01 and BH20-02. The boreholes were advanced through Frederick Street to depths ranging from 38.3 to 38.4 m.

Details of boreholes drilled during the previous and current site investigations and field testing, including location and termination depths are presented in Table 3.1.

It should be noted that no boreholes were drilled at the Pier as part of the current site investigation at the direction of MTO.

The approximate locations of the completed boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix C. The coordinates and elevations of the current and previous boreholes are given on the drawings and on the individual Record of Borehole Sheets in Appendices A and B, respectively.

The ground surface elevations and coordinates of the recent as-drilled boreholes were determined by Thurber using a Trimble R10 survey unit.

Prior to commencing the site investigation, utility clearances were obtained for all borehole locations.

During the current investigation, a truck-mounted B60 drill rig was used in conjunction with hollow-stem augers and tricone to advance the boreholes. In general, soil samples were obtained at selected intervals using a 50mm diameter split spoon sampler in conjunction with the Standard Penetration Testing (SPT).

The drilling, sampling and in-situ testing operations were supervised on a full-time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil samples for transport to Thurber's laboratory for further examination and testing. Results of field drilling and sampling of the current and previous investigations are presented on the Record of Borehole sheets in Appendices A and B, respectively.

Groundwater conditions in the open boreholes were observed throughout the drilling operations One standpipe piezometers were installed in Borehole BH20-01 to permit groundwater level monitoring. The piezometer consisted of a 25 mm Schedule 40 PVC pipe with a 3.0 m long slotted screen enclosed in a column of filter sand. to permit groundwater level monitoring. The completion details of the piezometers and boreholes are summarized in Table 3.1. The piezometer in Borehole BH20-01 was decommissioned in accordance with O.Reg. 903.

**Table 3.1 – Borehole Completion Details**

| Foundation Unit | Borehole | Ground Surface Elevation (m) | Borehole Depth / Base Elevation (m) | Piezometer Tip Depth /Elevation (m) | Completion Details |
|---|---|---|---|---|---|
| West Abutment | BH20-01 | 327.5 | 38.3/289.2 | 19.8/307.7 | Piezometer with 3.0 m slotted screen installed with sand filter from 19.8 m to 15.8 m, bentonite holeplug to 13.7 m, and grout from 13.7 m to surface |
| | 02 | 326.9 | 25.5/301.4 | None Installed | N/A |
| | 03 | 326.7 | 18.1/308.6 | None Installed | N/A |
| Pier | 07 | 325.9 | 17.0/308.9 | None Installed | N/A |
| | 10 | 325.5 | 17.5/308.0 | None Installed | N/A |
| East Abutment | BH20-02 | 325.0 | 38.4/286.6 | None Installed | Borehole backfilled with holeplug, to 0.1 m and asphalt cold patch to surface |
| | 14 | 325.3 | 20.3/305.0 | None Installed | N/A |

## 4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to grain size analysis and Atterberg Limits testing. All the laboratory tests were carried out in accordance with MTO and/or ASTM Standards, as appropriate. The results of the laboratory testing of current and previous investigations are summarized on the Record of Borehole sheets and figures in Appendices A and B, respectively.

In order to assess the potential for sulphate attack on concrete foundations, as well as the potential for corrosion associated with the structure, two samples were collected and submitted to SGS Canada Inc., a CALA accredited analytical laboratory in Lakefield, Ontario, for analytical testing of corrosivity parameters and sulphate content. The results of the analytical testing are summarized in Section 6 and are presented in Appendix A.

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## 5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendices A and B. Details of the encountered soil stratigraphy along the proposed alignment are presented in these appendices and on the "Borehole Locations and Soil Strata" drawings in Appendix C. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

In general, the subsurface conditions at the site consist of a pavement structure and layers of sand fill and sand overlying clayey silt and silty clay above a deposit of sandy silt to silty sand. The sandy silt to silty sand is underlain by a lower silty clay layer which is in turn underlain by a deposit of silty clay till.

A detailed description of the subsurface conditions is presented in the following sections.

### 5.1 Pavement Structure

Pavement structure consisting of asphalt overlying sand and gravel fill (road base) was encountered in the recent boreholes advanced through the Frederick Street road platform (i.e. BH20-01 and BH20-02)

The measured thickness of the asphalt ranged between 163 mm and 200 mm at the borehole locations. The sand and gravel fill underlying the asphalt was approximately 0.5 to 0.6 m thick.

### 5.2 Sand Fill

Sand fill was encountered underlying the sand and gravel fill (pavement structure) in BH20-01 and BH20-02.

The thickness of the fill ranged from approximately 1.6 m to 3.3 m and the base of the fill was encountered at depths of 2.3 m and 4.1 m (Elev. 323.4 to 322.7). The fill was generally described as gravelly to some gravel and contained some silt within the upper 1.4 to 1.6 m, and contained no gravel and trace silt below the upper 1.4 to 1.6 m.

The SPT 'N' values measured in the sand fill ranged from 1 to 28 blows per 0.3 m of penetration, indicating a highly variable very loose to compact relative density. The natural moisture contents measured on samples of the sand fill ranged from 3 percent to 6 percent.

The results of a grain size analysis test conducted on a sample of the sand fill are provided on the Record of Borehole Sheets in Appendix A and illustrated in Figure A1 in Appendix A. The results are summarized as follows:

| Soil Particles | (%) |
|---|---|
| Gravel | 0 |
| Sand | 89 |
| Silt + Clay | 11 |

### 5.3 Sand

A native sand deposit was encountered in all of the historic boreholes. The sand was encountered underlying topsoil in all of the historic boreholes with the exception of Borehole 10 where it was encountered underlying gravelly sand fill. It is expected that this layer was removed from beneath the bridge during construction of the KWE.

The thickness of the sand ranged from 2.8 m to 6.4 m and the depth to the base of the sand deposit ranged from 4.3 m to 7.9 m below ground surface (Elev. 321.6 to 317.4). The fill generally contained trace amounts of silt.

The SPT ‘N’ values measured in the sand ranged from 6 to 62 blows per 0.3 m of penetration, indicating a loose to very dense relative density (typically compact). The natural moisture contents measured on samples of the sand ranged from 3 percent to 29 percent.

The results of grain size analysis testing conducted on samples of the sand are provided on the historic Record of Borehole Sheets in Appendix B. The results are summarized as follows:

| Soil Particles | (%) |
|---|---|
| Gravel | 0 to 47 |
| Sand | 46 to 97 |
| Silt | 1 to 20 |
| Clay | 0 |

### 5.4 Clayey Silt

A clayey silt layer was encountered underlying the sand fill in BH20-01, underlying the sand deposit in historic boreholes 2, 3, 6, 7, and beneath the silty sand/sandy silt interlayer in boreholes 10 and 11. The clayey silt layer ranges from 0.9 to 2.7 m thick and the base of the layer was encountered at depths ranging from 6.1 m to 8.7 m (Elev. 319.3 to 318.5). Trace to some gravel and sand was noted in the clayey silt.

SPT ‘N’ values measured in the clayey silt ranged from 9 to 64 blows per 0.3 m of penetration, indicating a stiff to hard consistency (typically very stiff to hard). The natural moisture contents measured on samples of the clayey silt ranged from 9 percent to 19 percent.

The results of grain size analyses testing conducted on samples of the clayey silt are provided on the Record of Borehole Sheets in Appendices A and B. The results of one test performed as part of the current investigation are shown on Figure A2 in Appendix A. The results from both investigations are summarized as follows:

| Soil Particles | (%) |
|---|---|
| Gravel | 0 to 14 |
| Sand | 7 to 27 |
| Silt | 44 to 78 |
| Clay | 14 to 20 |

The results of Atterberg Limits testing conducted on samples of the clayey silt from the previous investigation are summarized below.

| | |
|---|---|
| Liquid Limit | 19 to 34 |
| Plastic Limit | 12 to 17 |
| Plasticity Index | 6 to 17 |

The above results indicate that the clayey silt is of low plasticity with a group symbol of CL-ML.

### 5.5 Silty Sand/Sandy Silt Interlayer

A silty sand/sandy silt interlayer was encountered underlying the clayey silt layer in historic boreholes 6 and 7, underlying the sand deposit in boreholes 10 and 11, and beneath the upper silty clay layer in borehole 14. The interlayer ranged from 0.9 m to 3.7 m in thickness and the base of the interlayer was encountered at depths between 6.1 m and 12.5 m below ground surface (Elev. 319.4 and 312.8).

SPT ‘N’ values measured in the silty sand/sandy silt interlayer ranged from 29 blows per 0.3 m of penetration to 100 blows per 0.09 m of penetration, indicating a compact to very dense relative density (typically very dense). The natural moisture contents measured on samples of the silty sand/sandy silt interlayer ranged from 11 percent to 18 percent.

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Grain size testing was performed on samples of the interlayer during the previous investigation. The results of the testing are shown on the Record of Borehole Sheets in Appendix B and summarized as follows:

| Soil Particles | (%) |
|---|---|
| Gravel | 0 |
| Sand | 25 to 67 |
| Silt | 32 to 75 |
| Clay | 0 |

### 5.6 Upper Silty Clay

A deposit of silty clay was encountered in all of the boreholes advanced at the site underlying the sand fill, clayey silt, and silty sand/sandy silt layers. The silty clay deposit ranged from 3.9 m to 10.3 m thick and the base of the deposit was encountered at depths between 11.9 m and 18.0 m below ground surface (Elev. 314.8 and 307.6). Trace sand was noted in the silty clay.

SPT ‘N’ values measured in the silty clay ranged from 24 to 194 blows per 0.3 m of penetration, indicating a very stiff to hard consistency (typically hard). The natural moisture contents measured on samples of the silty clay ranged from 13 percent to 40 percent. The results of grain size analyses testing conducted on samples of the silty clay are provided on the Record of Borehole Sheets in Appendices A and B and the results of grain size testing performed during the current investigation are shown on Figure A3 in Appendix A. A summary of the test results from both investigations is provided below:

| Soil Particles | (%) |
|---|---|
| Gravel | 0 |
| Sand | 0 to 1 |
| Silt | 30 to 54 |
| Clay | 45 to 70 |

The results of Atterberg Limits testing conducted on samples of the silty clay are shown on the Record of Borehole Sheets in Appendices A and B. The results of the testing from the current investigation are presented on Figure A7 in Appendix A. The results from both the current and previous investigations are summarized below. Based on the results, the silty clay is of intermediate to high plasticity (CI to CH).

| | |
|---|---|
| Liquid Limit | 33 to 55 |
| Plastic Limit | 15 to 23 |
| Plasticity Index | 18 to 37 |

### 5.7 Sandy Silt to Silty Sand

A deposit of sandy silt to silty sand containing trace clay was encountered underlying the upper silty clay deposit in all boreholes advanced at this site. The thickness of the sandy silt to silty sand deposit ranged from 0.7 m to 6.1 m and the base of the deposit was encountered at depths ranging from 17.8 m and 19.4 m below ground surface (Elev. 308.9 and 306.7). A number of boreholes were terminated in this layer.

SPT 'N' values measured in the sandy silt to silty sand ranged from 45 blows per 0.3 m of penetration to 123 blows per 0.15 m of penetration, indicating a dense to very dense relative density (typically very dense). The natural moisture contents measured on samples of the sandy silt to silty sand ranged from 10 percent to 31 percent.

The results of grain size analyses testing conducted on samples of the sandy silt to silty sand are provided on the Record of Borehole Sheets in Appendices A and B and the results of a grain size test performed during the current investigation are shown on Figure A4 in Appendix A. A summary of the test results from both investigations is provided below:

| Soil Particles | (%) |
|---|---|
| Gravel | 0 |
| Sand | 5 to 28 |
| Silt | 66 to 95 |
| Clay | 0 to 6 |

### 5.8 Lower Silty Clay

A relatively thick deposit of grey silty clay containing trace sand was encountered in the boreholes which penetrated the sandy silt to silty sand deposit (i.e. BH20-01, BH20-02, 2, and 14). Where fully penetrated, this lower silty clay deposit ranged in thickness from 14.4 to 14.5 m and the base of the layer was located between 32.3 m and 33.8 m depth (Elev. 293.7 and 292.7). Historic boreholes 2 and 14 were terminated in this deposit.

SPT 'N' values measured within the lower silty clay ranged from 23 to 130 blows per 0.3 m of penetration, indicating a very stiff to hard consistency. The natural moisture contents measured on samples of the lower silty clay ranged from 16 percent to 29 percent.

Grain size analyses were carried out on samples of the lower silty clay as part of the current investigation. The results of grain size analyses are provided on the Record of Borehole Sheets in Appendix A and illustrated in Figure A5 in Appendix A. The results are summarized as follows:

| Soil Particles | (%) |
|---|---|
| Gravel | 0 |
| Sand | 1 to 4 |
| Silt | 36 to 43 |
| Clay | 56 to 61 |

The results of Atterberg Limits testing conducted on samples of the lower silty clay are shown in Figure A8 in Appendix A and summarized below.

| | |
|---|---|
| Liquid Limit | 35 to 44 |
| Plastic Limit | 17 to 18 |
| Plasticity Index | 18 to 26 |

The results indicate that the silty clay is of medium plasticity with a group symbol of CI.

### 5.9 Lower Sand

A layer of sand with gravel and trace silt was encountered in BH20-02 underlying the lower silty clay deposit. The sand layer was 2.3 m thick and its base was at a depth of 34.6 m (Elev. 290.4).

An SPT ‘N’ value of 69 blows per 0.3 m of penetration was measured within the sand layer, indicating a very dense relative density. A natural moisture content of 10 percent weas measured on a sample of the sand.

### 5.10 Silty Clay Till

Silty clay till, sandy to with sand, trace gravel to gravelly, was encountered underlying the lower silty clay and sand layers in BH20-01 and BH20-02, respectively. The surface of the till was encountered at depths ranging from 33.8 m to 34.6 m (Elev. 293.7 to 290.4). BH20-01 and BH20-01 were terminated in this till deposit at depths between 38.3 m and 38.4 m.

SPT ‘N’ values measured within the till ranged from 76 blows per 0.25 m of penetration to 105 blows per 0.17 m of penetration, indicating a hard consistency. The natural moisture contents measured on samples of the till ranged from 9 percent to 27 percent.

The results of grain size analyses conducted on samples of the till are provided on the Record of Borehole Sheets in Appendix A and illustrated in Figure A6 in Appendix A. The results are summarized as follows:

| Soil Particles | (%) |
|---|---|
| Gravel | 3 to 21 |
| Sand | 31 to 36 |
| Silt | 28 to 51 |
| Clay | 15 |

The results of an Atterberg Limits test conducted on a sample of the till are shown in Figure A9 in Appendix A and summarized below.

| | |
|---|---|
| Liquid Limit | 19 |
| Plastic Limit | 11 |
| Plasticity Index | 8 |

The results indicate that the silty clay till is of low plasticity with a group symbol of CL.

### 5.11 Groundwater Conditions

A monitoring well was installed in BH20-01 to permit monitoring of the water level. Water levels were observed in the current and historic boreholes during and upon completion of drilling. The water levels measured in the piezometers and upon completion of drilling are summarized below.

**Table 5.4 – Water Level Measurements**

| Foundation Unit | Borehole | Date | Water Level (m) | | Comment |
|---|---|---|---|---|---|
| | | | Depth | Elevation | |
| West Abutment | BH20-01 | August 24, 2020 | 5.5 | 322.0 | Piezometer |
| | 02 | May 26, 1966 | 4.0 | 322.9 | Open Borehole |
| | 03 | May 31, 1966 | 3.9 | 322.8 | Open Borehole |
| Pier | 07 | May 31, 1966 | 3.2 | 322.7 | Open Borehole |
| | 10 | June 3, 1966 | 3.5 | 322.0 | Open Borehole |

| | | | | | |
|---|---|---|---|---|---|
| East Abutment | BH20-02 | August 21, 2020 | 2.3 | 322.7 | Open Borehole |
| | 14 | June 2, 1966 | 4.0 | 321.3 | Open Borehole |

The groundwater levels measured in the piezometers and open boreholes ranged from 2.3 m to 5.5 m below the ground surface (Elev. 322.9 to 321.3). It is noted that the water levels at the site were either measured behind the abutments or prior to cut excavation for the KWE. Therefore, no piezometers were installed through the KWE lanes to determine the groundwater level beneath the bridge. In general, the groundwater level beneath the bridge is expected to be located slightly below the highway grade (i.e. at or below Elev. 320 m).

The above values are short-term readings, and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

## 6 CORROSIVITY AND SULPHATE TEST RESULTS

Soil samples from Boreholes BH 20-01 and BH 20-02 were submitted for analytical testing of corrosivity parameters and sulphate. The results of the analytical tests are shown in Table 6.1. The laboratory certificates of analysis are presented in Appendix A.

**Table 6.1 – Analytical Test Results**

| Parameter | Units (Soil) | Test Results | |
|---|---|---|---|
| | | BH20-01 SS 4 Depth 3.4 m (Sand Fill) | BH20-02 SS 3 Depth 2.6 m (Native Sand) |
| Sulphide | % | <0.04 | <0.04 |
| Chloride | µg/g | 210 | 750 |
| Sulphate | µg/g | 8.3 | 21 |
| pH | No unit | 9.66 | 9.37 |
| Electrical Conductivity | µS/cm | 547 | 1120 |
| Resistivity | Ohms.cm | 1830 | 892 |
| Redox Potential | mV | 287 | 285 |

## 7 MISCELLANEOUS

Landshark Drilling of Brantford, Ontario supplied a B60 truck-mounted drill rig and conducted the drilling, sampling and in-situ testing operations for the present investigation.

The coordinates and elevations for the boreholes were obtained by Thurber using a Trimble R10.

The drilling and sampling operations in the field for the current investigation were supervised on a full-time basis by Thurber field technicians.

Geotechnical laboratory testing was carried out at Thurber's geotechnical laboratory. Analytical laboratory testing was carried out by SGS Canada Inc.

Overall supervision of the field program, interpretation of the data, and preparation of the report was conducted by Mr. Geoff Lay, P.Eng. Mr. Jason Lee, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations projects, reviewed the report.

Thurber Engineering Ltd


Geoff Lay, P.Eng.
Geotechnical Engineer


Jason Lee, P.Eng.
Principal/Senior Geotechnical Engineer

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LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
02/09/2021
PROVINCE OF ONTARIO

P.K. Chatterji, P.Eng.,
Review Principal, Designated MTO Contact

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**FOUNDATION INVESTIGATION AND DESIGN REPORT**
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**HIGHWAY 7- NEW, KITCHENER TO GUELPH**
**G.W.P. 408-88-00**

**GEOCRES NO. 40P8-285**

**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

## 8 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system for the proposed structure that will carry the eastbound lanes (EBL) and westbound lanes (WBL) of Frederick Street over the Kitchener-Waterloo Expressway (KWE) in the Regional Municipality of Waterloo, Ontario.

Based on the GA drawing provided by WSP, dated February 2019, the proposed structure is a 92.1 m long two-span structure supported on caissons. The structure will be constructed along the same alignment of the existing bridge but is longer than the existing bridge in order to accommodate the proposed Bruce Street ramp and widened E-S ramp. The grade at the west and east abutments will be raised approximately 1 and 1.5 m to Elev. 328.6 m and 326.6 m, respectively. Retaining walls will be constructed at all four quadrants of the bridge.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. The contractors must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects, which could affect the design of the project. Contractors must make their own interpretation of the information provided as it may affect equipment selection, proposed construction methods and scheduling.

The discussion and recommendations presented in this report are based on the information provided by WSP and on the factual data obtained in the course of the previous and the present investigations.

It is noted that this report does not address the design of the retaining walls, which are addressed in the following report:

- Foundation investigation report for Proposed Retaining Walls, Highway 7-New, Kitchener to Guelph, G.W.P. 408-88-00, Geocres No. 40P9-58, prepared by Thurber Engineering Ltd., dated May 6, 2020

## 9 STRUCTURE CLASSIFICATION

In accordance with the currently applicable Canadian Highway Bridge Design Code (CHBDC) (2019) CSA S6-19, the analysis and design of structures are influenced by its importance category and consequence classification. Such designations are defined by the Regulatory Authority which, in this case, is the Ministry of Transportation of Ontario (MTO).

For the purpose of reporting, this structure has been classified as a Major-Route Bridge with Typical Consequence based on CHBDC S6-19 Sections 4.4.2 and 6.5.2, respectively.

Based on the above classification and Table 6.1 in Section 6.5.2 in the CHBDC (2019), a consequence factor, $\psi$, of 1.0 has been used for assessing ULS and SLS factored geotechnical resistances. Should the consequence classification change, the geotechnical assessment and recommendations will need to be reviewed and revised as necessary.

## 10 STRUCTURE FOUNDATIONS

In general, the subsurface conditions at the site consist of a pavement structure and layers of sand fill and sand overlying clayey silt and silty clay above a deposit of sandy silt to silty sand. The sandy silt to silty sand is underlain by a lower silty clay layer which is in turn underlain by a deposit of silty clay till.

The groundwater levels measured in the piezometers and open boreholes ranged from 2.3 m to 5.5 m below the ground surface (Elev. 322.9 to 321.3). It is noted that the water levels at the site were either measured behind the abutments or prior to cut excavation for the KWE. Therefore, no piezometers were installed through the KWE lanes to determine the groundwater level beneath the bridge. In general, the groundwater level beneath the bridge is expected to be located slightly below the highway grade (i.e. at or below Elev. 320 m).

In the preparation of the geotechnical design recommendations, consideration was given to the following foundation types:

1. Spread footings on native soil
2. Spread footings on engineered fill
3. Augered caissons (drilled shafts) with steel casings
4. Steel H-piles driven into the hard/very dense soils

A comparison of the foundation alternatives based on advantages and disadvantages of each is included in Appendix D.

Spread footings founded on native soil may be considered to support the abutments and pier. This option would require excavations in conjunction with roadway protection for KWE travelled lanes.

Spread footings founded on an engineered fill pad are considered feasible but would require relatively large excavations for engineered fill pad construction. Due to space constraints beneath the bridge, this option has not been developed further.

The GA drawing indicates that augured caissons (drilled shafts) have been selected to support the abutments and pier. The caissons can be designed to be structurally connected to the superstructure without a pile cap and are therefore preferrable at the pier location where space is restricted between the travelled lanes. This option will require the use of temporary liners and drilling mud to support the caisson sidewalls in the cohesionless deposits below the groundwater table. This option may also require placement of concrete using tremie methods.

Alternatively, consideration may be given to supporting the abutments and pier on driven steel H-piles. This option would require installation of a roadway protection system and excavation at the pier to permit pile cap construction. It is noted that pile driving will produce vibrations which may be disruptive to residents in the area and could damage nearby structures and utilities. Vibration monitoring and a pre-condition survey of existing structures and utilities should be carried out if driven piles are selected for this site.

## 10.1 Spread Footings on Native Soil

Based on the subsurface conditions encountered at this site, consideration may be given to supporting the abutments and pier of the new structure on spread footings bearing on competent undisturbed native soil. There would be a risk of disturbing the subgrade during footing removal with this option which could result in additional settlement of the new bridge footing.

The highest founding levels and corresponding geotechnical resistances recommended at each foundation unit, based on the borehole data, are presented in the following table. Footings should be founded at or below these elevations, subject to minimum requirements for frost protection. The geotechnical resistances provided are based on an assumed minimum footing width of 3 m.

**Table 10.1 – Highest Recommended Founding Levels**

| Location | Borehole | Minimum Depth below KWE highway grade (m) | Highest Recommended Founding Level (m) | Founding Soil Type | Factored Geotechnical Resistance | |
|---|---|---|---|---|---|---|
| | | | | | ULS (kPa) | SLS (kPa) |
| West Abutment | BH20-01<br>02<br>03 | 1.4 | 318.5 | Very Stiff to Hard Silty Clay | 450 | 300 |
| Pier | 07<br>10 | 1.4 | 318.0 | Very Dense Sandy Silt to Silty Sand / Very Stiff to Hard Silty Clay | 450 | 300 |
| East Abutment | BH20-02<br>14 | 1.4 | 317.5 | Very Stiff to Hard Silty Clay | 450 | 300 |

The values of the Factored Geotechnical Resistance at ULS were assessed assuming a Consequence Factor equal to 1 (Typical), and a Resistance Factor equal to 0.5 (Typical degree of understanding of the subsurface conditions), as per CHBDC 2019. The Geotechnical Resistance at SLS was assessed assuming a factor of 0.8 for typical degree of understanding of the subsurface conditions.

The bearing resistances in Table 10.1 are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be adjusted as shown in the CHBDC (2019) Clauses 6.10.2 to 6.10.5.

The geotechnical SLS values given above are based on an estimated total settlement not exceeding 25 mm. Any settlement is expected to be substantially complete by the end of construction. Differential settlement is not expected to exceed 20 mm across the width of the structure or between foundation elements.

The sliding resistance of cast-in-place concrete founded on the native sandy silt/silty sand and the native silty clay should be designed using ultimate unfactored coefficients of base friction of 0.45 and 0.4, respectively. Resistance factors of 0.6 and 0.8 should be used when checking lateral stability of the footings sliding on cohesive and cohesionless soils, respectively.

The footing excavations are expected to extend below the groundwater level. Local groundwater control and prior dewatering, as discussed in Section 14, will be required to construct the footing in the dry and to prevent disturbance of the footing base.

Demolition of the existing footings should minimize disturbance to the subgrade. The bases of the foundation excavations should be inspected by a geotechnical engineer to confirm that the exposed subgrade surface conforms to the design requirements and has been adequately prepared to receive concrete. Once approved, the subgrade should be protected by a working mat with a minimum thickness of 100 mm and consisting of mass concrete of the same strength and class as that of the footing. Where subexcavation is required to remove unsuitable material from below the design founding level, the founding surface should be re-established using the same mass concrete.

## 10.2 Augured Caissons (Drilled Shafts) with Steel Casings

The GA drawing indicates that the abutments and pier will be supported on caissons. Caissons are considered feasible at this site and may be founded within the very dense silty sand to sandy silt or the hard silty clay till deposit.

### 10.2.1 Axial Resistance

Table 10.2 presents the factored axial geotechnical resistances calculated for 0.9 m and 2.1 m diameter caissons. It should be noted that no boreholes were drilled at the pier as part of the current site investigation at the direction of MTO. The axial resistances provided for the pier are based on the existing info from BH07 & BH10 and extrapolation of the subsurface conditions encountered in the deep boreholes advanced at the abutments (i.e. BH20-01 and BH20-02).

**Table 10.2 – Estimated Axial Resistance and Pile Tip Elevation for Caisson**

| Foundation Unit | Borehole | Approx. Underside Pile Cap Elev. (m) | Approx. Caisson Base Elev. (m) | Minimum Caisson Length Assumed (m) | Founding Strata | 0.9 m diameter | | 2.1 m diameter | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Factored ULS (kN) | Factored SLS** (kN) | Factored ULS (kN) | Factored SLS** (kN) |
| West Abutment | BH20-01<br>02<br>03 | 325.5 | 291 | 35 | Hard Silty Clay Till | 4,500 | 3,500 | - | - |
| Pier | 07<br>10 | - | 289.5 | 30 | Hard Silty Clay Till | - | - | 12,000(*) | 10,000(*) |
| East Abutment | BH20-02<br>14 | 323.0 | 288 | 35 | Hard Silty Clay Till | 4,500 | 3,500 | - | - |

**Corresponding to not more than 15 mm settlement.

(*)Based on extrapolation of the subsurface conditions encountered in the deep boreholes advanced at the abutments (i.e. BH20-01 and BH20-02).

The resistance values provided in Table 10.2 above are based on end bearing and shaft friction, assuming that the walls and base of each caisson are cleaned of loose material prior to placement of concrete.

It is noted, for larger diameter caissons, that it takes greater vertical movement at the base to fully mobilize the available geotechnical resistance. As such, the increase in SLS values (for a maximum 25 mm settlement) is not necessarily proportional to the increase in the corresponding ULS values.

The structural designer must check the structural capacities of the caissons against the geotechnical resistances.

### 10.2.2 Downdrag

Downdrag on the caissons is not expected to be an issue at this site.

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### 10.2.3 Lateral Resistance in Soil

The geotechnical lateral resistance of a caisson may be calculated using the coefficient of horizontal subgrade reaction ($k_s$) and the ultimate lateral resistance ($P_{ult}$) as follows:

Clayey Silt, Silty Clay, Silty Clay Till (cohesive soils)

$k_s$ = $67\ C_u / B$ ($kN/m^3$)
$p_{ult}$ = $9\ C_u$ (kPa) at and below a depth of 3B reduced to zero at ground surface

where $p_{ult}$ = ultimate lateral resistance mobilized by a caisson, kPa
$C_u$ = undrained shear strength of cohesive soils, kPa
$\gamma$ = unit weight of soil, $kN/m^3$
B = diameter of caisson, m

Sand, Sandy Silt to Silty Sand (cohesionless soils)

$k_s$ = $n_h \cdot z / B$ ($kN/m^3$)
$p_{ult}$ = $3 \cdot \gamma' \cdot z \cdot K_p$ (kPa)
where z = depth of embedment of caisson, m
B = diameter of caisson, m
nh = coefficient related to soil density, $kN/m^3$ , Table 10.5
$\gamma'$ = Buoyant unit weight of soil, kN/m3, Table 10.5
Kp = passive earth pressure coefficient, Table 10.5

The above equations and parameters provided in Table 10.3 may be used to analyze the interaction between a caisson and the surrounding soil. The lateral pressure obtained from the analysis should not exceed the ultimate lateral resistance.

**Table 10.3 – Recommended Geotechnical Parameters for Lateral Resistance Design**

| Location | Reference Boreholes | Approx. Elevation (m) | | Undrained Shear Strength $C_u$ (kPa) | Unit Weight $\gamma$ ($kN/m^3$) | $K_p$ | $n_h$ ($kN/m^3$) | Soil Conditions |
|---|---|---|---|---|---|---|---|---|
| | | Top | Bottom | | | | | |
| West Abutment | BH20-01<br>02<br>03 | Top of Pile | 320 | - | 9 | 3.0 | 2,500 | Existing Fill |
| | | 320 | 318.5 | 90 | 10 | - | - | Clayey Silt - Stiff to Hard |

| Location | Reference Boreholes | Approx. Elevation (m) | | Undrained Shear Strength $C_u$ (kPa) | Unit Weight $\gamma$ ($kN/m^3$) | $K_p$ | $n_h$ ($kN/m^3$) | Soil Conditions |
|---|---|---|---|---|---|---|---|---|
| | | Top | Bottom | | | | | |
| | | 318.5 | 314.5 | 150 | 11 | - | - | Upper Silty Clay - V. Stiff to Hard |
| | | 314.5 | 308 | | 12 | 4.2 | 10,900 | Sandy Silt to Silty Sand - V. Dense |
| | | 308 | 293.5 | 175 | 11 | - | - | Lower Silty Clay - V. Stiff to Hard |
| | | 293.5 | Below 289.0 | 250 | 11.5 | - | - | Silty Clay Till - Hard |
| Pier | 07<br>10 | Top of Pile | 319 | 90 | 10 | - | - | Clayey Silt - Stiff to Hard |
| | | 319 | 318 | - | 12 | 3.6 | 6,800 | Sandy Silt to Silty Sand Interlayer - Dense to V. Dense |
| | | 318 | 309.5 | 200 | 11 | - | - | Upper Silty Clay - V. Stiff to Hard |
| | | 309.5 | Below 308.0 | - | 12 | 4.2 | 10,900 | Sandy Silt to Silty Sand - V. Dense |
| East Abutment | BH20-02<br>14 | Top of Pile | 318 | - | 9 | 3.0 | 2,500 | Native Sand |
| | | 318 | 311 | 150 | 11 | - | - | Upper Silty Clay - V. Stiff to Hard |
| | | 311 | 307 | - | 12 | 4.0 | 9,000 | Sandy Silt to Silty Sand - V. Dense |
| | | 307 | 292.5 | 175 | 11 | - | - | Lower Silty Clay - V. Stiff to Hard |

| Location | Reference Boreholes | Approx. Elevation (m) | | Undrained Shear Strength $C_u$ (kPa) | Unit Weight $\gamma$ (kN/m$^3$) | $K_p$ | $n_h$ (kN/m$^3$) | Soil Conditions |
|---|---|---|---|---|---|---|---|---|
| | | Top | Bottom | | | | | |
| | | 292.5 | 290.5 | - | 12 | 3.8 | 8,000 | Sand - V. Dense |
| | | 290.5 | Below 286.5 | 250 | 11.5 | - | - | Silty Clay Till - Hard |

* Bouyant unit weight below water table

The spring constant, Ks, for analysis may be obtained by the expression, Ks = ks L D (kN/m), where ks is the coefficient of horizontal subgrade reaction (kN/m3), D is the caisson diameter (m) and L is the length (m) of the caisson segment or element used in the analysis. The ultimate lateral resistance, Pult, can be obtained from the expression, Pult = pult L D. This represents the ultimate load at which the supporting soil fails and will not support any additional load at greater displacements.

The coefficient of horizontal subgrade reaction and ultimate lateral resistance should be reduced based on the caisson/pile spacing to account for group effect. The group efficiency factors provided in CHBDC (2019) Commentary Section C6.11.3.4 may be used for a caisson/pile group oriented perpendicular or parallel to the direction of loading. The group efficiency factors can be calculated based on side-by-side and line-by-line factors shown in Figures C6.22, C6.23 and C6.24 of the CHBDC (2019), S6:19 (Commentary).

### 10.2.4 Caisson Installation

Caissons must be installed in accordance with OPSS.PROV 903 and SP 109F57 where applicable.

The caisson installation equipment should be able to dislodge and remove any obstructions such as cobbles and boulders and penetrate the silty clay till. An NSSP addressing this issue must be included in the contract documents to alert the bidders. Suggested wording for such an NSSP is provided in Appendix E.

Based on water level measurements at the site, the groundwater table is located approximately 2.3 m and 5.5 m below existing ground surface (Elev. 322.9 m and 321.3 m). Soil sloughing and water seepage will occur in unsupported holes primarily within the sand fill, sand, and silty sand/sandy silt layers. Therefore, construction of caissons will require the use of temporary steel liners to support the caisson sidewalls and to provide seepage cut-off where required. Synthetic slurry should be used to balance hydrostatic head and to prevent basal heave.

The Contractor should use appropriate means such as a cleanout bucket, air lift, hydraulic pump, or other devices approved by Engineer to clean the bottom of the excavation of all shafts. The Contractor should also inspect the bottom of the excavated shaft using a waterproof downhole colour camera (e.g., Drilled Shaft Inspection Device or SID), or a quantitative measurement of base sediment (e.g., Shaft Quantitative Inspection Device or SQUID), or an approved alternate to verify base cleanliness. The base of all shafts should be clear of any base sediment at the time of concreting to ensure direct contact between the concrete and subgrade.

Any accumulated water within the hole may have to be pumped out prior to placing concrete. Concrete should be placed with a minimum delay (i.e. no more than 6 hours) after each caisson is drilled, cleaned and inspected. If accumulated water in the caisson hole cannot be removed, consideration should then be given to using the tremie technique to place concrete inside the caisson hole.

Suggested wording for an NSSP addressing caisson construction is provided in Appendix E.

### 10.3 Driven Steel H-Piles

Consideration may be given to supporting the bridge abutments and pier on steel H-piles driven into the hard silty clay till deposit.

#### 10.3.1 Axial Resistance

The axial resistances of HP 310 X 110 and HP 360 x 132 steel piles driven into the hard silty clay till deposit are provided in Table 10.4 below. It should be noted that no boreholes were drilled at the pier as part of the current site investigation at the direction of MTO. The axial resistances provided for the pier are based on the existing info from BH07 & BH10 and extrapolation of the subsurface conditions encountered in the deep boreholes advanced at the abutments (i.e. BH20-01 and BH20-02).

**Table 10.4 – Estimated Axial Resistance and Pile Tip Elevation for Driven H-Piles**

| Foundation Unit | Borehole | Approx. Underside Pile Cap Elev. (m) | Approx. Pile Tip Elev. (m) | Minimum Pile Length Assumed (m) | Founding Strata | Pile Section HP 310 X 110 | | Pile Section HP 360 X 132 | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Factored ULS (kN) | Factored SLS (kN) | Factored ULS (kN) | Factored SLS (kN) |
| West Abutment | BH20-01<br>02<br>03 | 325.5 | 291 | 34.5 | Hard Silty Clay Till | 1,600 | 1,400 | 1,800 | 1,600 |
| Pier | 07<br>10 | 319.5 | 289.5 | 30 | Hard Silty Clay Till | 1,600(*) | 1,400(*) | 1,800(*) | 1,600(*) |
| East Abutment | BH20-02<br>14 | 323.0 | 288 | 35 | Hard Silty Clay Till | 1,600 | 1,400 | 1,800 | 1,600 |

(*)Based on extrapolation of the subsurface conditions encountered in the deep boreholes advanced at the abutments (i.e. BH20-01 and BH20-02).

The values of the Factored Geotechnical Resistance at ULS were assessed assuming a Consequence Factor equal to 1 (Typical), and a Resistance Factor equal to 0.4 (Typical degree of understanding of the subsurface conditions), as per CHBDC 2019. The SLS values correspond to a maximum pile settlement of 25 mm. The Factored Geotechnical Resistance at SLS was assessed assuming a factor of 0.8 for typical degree of understanding of the subsurface conditions.

The structural resistance of the pile must be checked by the structural designer.

### 10.3.2 Downdrag

Downdrag on the piles is not expected to be an issue at this site.

### 10.3.3 Lateral Resistance

For pile lateral resistance design, soil-pile interaction analyses may be carried out using the coefficient of horizontal subgrade reaction values provided in Table 10.3.

The methodology outlined previously in Section 10.2.3 may be used to estimate the lateral geotechnical resistance of the pile by substituting the caisson diameter, D with the pile width, B.

### 10.3.4 Pile Installation

All piles shall be installed in accordance with OPSS.PROV 903 and SP 109F57.

Pile driving must be controlled in accordance with Standard Provision SS103-11 (Hiley Formula) and an ultimate pile resistance must be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The Hiley formula does not need to be used until the pile tip is within 2 m of the design tip elevation. The appropriate pile driving note to be shown on the contract drawing is “Piles to be driven in accordance with Standard SS103-11 using an ultimate geotechnical resistance of R kN per pile”. “R” must have a minimum value of twice the factored design load at ULS. It is recommended that Pile Driving Analysis (PDA) testing be conducted on at least 2 piles per foundation element in conjunction with the Hiley tests at this site, to establish set criteria, ensure the integrity of the pile and verify pile ultimate geotechnical resistance.

To facilitate pile installation, embankment fill through which piles will be driven must not contain any material with particle sizes greater than 75 mm.

Hard driving conditions through the hard/very dense soils should be expected. Cobbles and boulders should also be anticipated within the silty clay till deposit which may affect pile installation. In order to minimize pile damage while driving the piles hard/dense zones, cobbles, and boulders, to achieve the required tip elevations and soil resistance, it is recommended that the pile tips be reinforced with Titus steel (Standard H-point).The Contract Documents must contain a NSSP alerting the Bidders to the presence of hard/dense zones, cobbles, and boulders, and the use of PDA Testing. Suggested texts for the NSSP’s are included in Appendix E. The NSSP should contain a requirement to terminate driving before the pile is damaged by overdriving.

Vibrations produced during pile driving may disrupt nearby residents and damage nearby structures and utilities. A preconstruction condition survey of existing structures and utilities should be carried out prior to commencement of pile installation. Vibration monitoring should also be carried out during pile driving to limit potential impacts on existing facilities, and conditions carefully monitored for signs of disturbance.

It is understood that the City of Kitchener does not provide limits on vibration levels. Therefore, it is recommended that the vibration levels stipulated in the City of Toronto By-law 514-2008 be adopted for this project. The limits are provided in the table below.

**Table 10.5 – Vibration Limits**

| Vibration Frequency (Hz) | Vibration Peak Particle Velocity (mm/s) |
|---|---|
| Less than 4 | 8 |
| 4 to 10 | 15 |
| More than 10 | 25 |

### 10.4 Abutment Design Considerations

From a geotechnical perspective, the conditions at this site are considered to be suitable for the design of conventional, semi-integral or integral abutments.

For integral abutments, the flexibility of the upper portion of the pile may be provided by a single corrugated steel pipe (CSP) system. Reference should be made to the integral abutment manual for details of this system. Piles should be driven first before pouring in loose uniform sand.

### 10.5 Frost Cover

The design depth of frost penetration for this site is 1.4 m as per OPSD 3090.101. All footing bases and undersides of pile caps/abutment stems must be provided with at least 1.4 m of soil cover as protection again frost action.

### 10.6 Demolition of Existing Structure

It is understood that the existing bridge will be closed for the project duration and that staged construction will not be required to maintain the live lanes of Frederick Street. The demolition of the existing abutment and pier footings must be carried out using procedures that minimize the potential for disturbance of the footing subgrade or undermining of the KWE travelled lanes.

Vibration monitoring should be carried out during structure demolition to confirm that the vibration levels (i.e. PPV) are within tolerable limits. Typical vibration limits are provided in Table 10.5.

### 10.7 Recommended Foundation

From a geotechnical perspective, the recommended foundations to support the abutments and pier are caissons founded within the very dense sandy silt to silty sand or hard silty clay till.

Steel H-piles are considered technically feasible but may not be preferrable considering the vibrations and noise produced during pile driving.

Spread footings are also considered feasible however there is a risk of subgrade disturbance during demolition of existing footings which could result in additional settlement of the new spread footings.

## 11 LATERAL EARTH PRESSURES

Earth pressures acting on the abutments may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC 2019 but are generally given by the expression:

$$p_h = K(\gamma h + q)$$

where: $p_h$ = horizontal pressure on the wall at depth h (kPa)

$K$ = earth pressure coefficient (see Table 11.1)

$\gamma$ = unit weight of retained soil (see Table 11.1)

$h$ = depth below top of fill where pressure is computed (m)

$q$ = value of any surcharge (kPa).

In accordance with Clause 6.12.3 of the CHBDC 2019, a compaction surcharge should be added. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS.PROV 501.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 11.1.

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**Table 11.1 – Earth Pressure Coefficients**

| Wall Condition | Earth Pressure Coefficient (K) | | | |
|---|---|---|---|---|
| | OPSS Granular A or OPSS Granular B Type II $\phi$ = 35°, $\gamma$ = 22.8 kN/m$^3$ | | OPSS Granular B Type I $\phi$ = 32°, $\gamma$ = 21.2 kN/m$^3$ | |
| | Horizontal Surface Behind Wall | Sloping Backfill (2H:1V) | Horizontal Surface Behind Wall | Sloping Backfill (2H:1V) |
| Active (Unrestrained Wall) | 0.27 | 0.40 | 0.31 | 0.48 |
| At rest (Restrained Wall) | 0.43 | 0.62 | 0.47 | 0.70 |
| Passive (Movement Towards Soil Mass) | 3.7 | - | 3.2 | - |

Note: Submerged unit weight should be used below the groundwater level.

If some movement of the wall is allowed (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. For rigid walls, at-rest horizontal earth pressures should be used.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) is preferred as it results in lower earth pressures acting on the wall.

The factors in Table 11.1 are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to be used in the design can be estimated from Figure C6.27 in the Commentary to the CHBDC 2019.

It is recommended that perforated sub-drains and/or weep holes be installed, where applicable, to provide positive drainage of the granular backfill behind the abutment walls. Reference may be made to OPSD 3101.150 where appropriate.

## 12 BRIDGE APPROACHES

Based on the GA drawing, the grade at the west and east abutments will be raised approximately 1 and 1.5 m to Elev. 328.6 m and 326.6 m, respectively.

Slope inclinations not steeper than 2 horizontal to 1 vertical (2H:1V) may be used for grade raise provided the embankments are constructed with granular or clean earth fill which does not contain medium or high plastic clay. All embankment fill must be constructed with adequate quality control in accordance with OPSS.PROV 206 and OPSS.PROV 501 requirements.

Prior to fill placement, the subgrade must be adequately prepared to receive the new fill. All vegetation, topsoil, organics, soft/loosened or wet soils should be sub-excavated.

It is also recommended that all permanent and temporary slope surfaces be vegetated and seeded in accordance with current MTO practice with reference to OPSS.PROV 804. Surface runoff and precipitation must be prevented from flowing perpendicularly down any slope surface. Erosion protection measures should be provided as necessary to maintain slope stability.

The foundation settlement under the approximately 1 to 1.5 m grade raise is expected to be negligible. Embankment settlement due to fill compression is expected to be negligible.

## 13 TEMPORARY EXCAVATION AND ROADWAY PROTECTION

The existing retaining walls and abutments beneath the bridge deck will be excavated to permit construction of the proposed S-E ramp (Bruce Street ramp) to the east and widening of the existing E-S ramp to the north. The cuts will extend to depths ranging from about 6 to 6.5 m from Frederick Street road grade primarily through sand and sand fill.

All excavations at this site must be carried out in accordance with the Occupational Health and Safety Act (OHSA). The excavation and backfilling for foundations must be carried out in accordance with OPSS.PROV 902.

For the purposes of the OHSA, the existing fills and native soils within the proposed excavation depth may be classified as Type 3 and Type 4 above and below the water table, respectively.

Temporary excavations for spread footings at the abutments will extend through the approach fills up to 10 m below the existing grade on Frederick Street. Temporary excavations for spread footing or pile cap construction at the pier will extend approximately 1.5 to 2.5 m below the existing grade on the KWE. The excavations will extend below the measured groundwater level.

Installation of roadway protection systems will be required to permit footing or pile cap construction at the pier location given the proximity of the pier to the KWE travelled lanes. Sloped excavations can be used at the north and south abutments provided that the adjacent lanes of the KWE can be closed or shifted to accommodate the temporary excavations for demolition of the existing structure footings.

The temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Construction Specification for Temporary Protection Systems) and designed for Performance Level 2. Based on available subsurface information, a shoring system consisting of a steel sheet pile wall or solider pile and lagging may be considered. Temporary shoring should be designed by a licensed Professional Engineer experienced in design of shoring with consideration of adjacent traffic loads and any sloping retained surfaces.

The following soil parameters may apply for design of the temporary protection systems with horizontal backfill:

| | | |
|---|---|---|
| $\gamma$ | = | 20 kN/m$^3$ |
| $\gamma_w$ | = | 10 kN/m$^3$ |
| $K_a$ | = | 0.35 (clayey silt) |
| | = | 0.27 (Upper sandy silt to silty sand) |
| | = | 0.31 (silty clay) |
| | = | 0.25 (Lower sandy silt to silty sand) |
| $K_p$ | = | 2.9 (clayey silt) |
| | = | 3.7 (Upper sandy silt to silty sand) |
| | = | 3.3 (silty clay) |
| | = | 4.0 (Lower sandy silt to silty sand) |

Excavations should regularly be inspected for evidence of instability if they have been left open for extended periods of time and following periods of heavy rain or thawing. If required, remedial actions must be taken to ensure the stability of the excavation and the safety of workers.

The selection of the method of excavation and the design of the shoring system is the responsibility of the contractor and must be based on his equipment, experience and interpretation of the site conditions. Provision must be made for the handling of pavement materials and potential obstructions in the fill.

## 14 GROUNDWATER AND SURFACE WATER CONTROL

The groundwater levels measured in the piezometers and open boreholes ranged from 2.3 m to 5.5 m below the ground surface (Elev. 322.9 to 321.3). It is noted that the water levels at the site were either measured behind the abutments or prior to cut excavation for the KWE( i.e.no piezometers were installed beneath the bridge through the KWE lanes to determine the groundwater level within the cut). In general, the groundwater level beneath the bridge is expected to be located slightly below the highway grade (i.e. at or below Elev. 320 m). The excavations for footing construction will extend to Elevation 317.5 to 318.5 or approximately 1.5

to 5.5 m below the anticipated groundwater level. Seasonal fluctuations of the groundwater level are to be expected.

Excavation for footing/pile caps construction will extend below the groundwater level. Seepage or perched water from the granular layers is to be expected. Excavation of the cohesionless native soils below the groundwater level without prior dewatering is not recommended since the inflow of groundwater will cause boiling and sloughing of the soil below the water table making it difficult to maintain a dry, sound base on which to work. Suitable systems that might be considered to maintain an unwatered condition at this site include pumping from filtered sumps for nominal penetration below the groundwater level, sheeted excavation (cofferdam) or vacuum well-points for deeper excavations. The dewatering system must be effective to maintain the water level at a minimum depth of 0.5 m below the final footing/pile cap grade throughout construction.

Dewatering of all excavations should be carried out in accordance with OPSS. PROV 517, SP 517F01 Amendment to OPSS 517, November 2016 (issued July 2017), NSP FOUN0003 and OPSS. PROV 902 and SP 109S12. It is recommended that a Professional Engineer with greater than 5 years of experience in designing dewatering systems be retained by the Contractor.

The design of the dewatering system is the responsibility of the Contractor, and the Contract Documents must alert him to this responsibility.

The groundwater and surface runoff must be controlled during construction to maintain a stable excavation and to allow concrete to be placed in a dewatered excavation. Placement of concrete must be done in the dry. Dewatering must remain operational and effective until the foundations are constructed and backfilled. Suggested wording for an NSSP in the regard is included in Appendix E.

## 15 BACKFILL TO ABUTMENTS

For backfilling immediately behind the new north and south abutment walls, it is recommended that the new fill be Granular A or Granular B Type II materials meeting the gradation and relevant requirements stipulated in OPSS.PROV 1010. Beyond this zone, clean earth fill may be used. The earth fill should not contain medium or high plastic clays or deleterious materials and organics.

The backfill should be in accordance with OPSS.PROV 206 requirements and OPSD 3101.150. Compaction equipment to be used adjacent to abutments/retaining structures must be restricted in accordance to OPSS.PROV 501.

The design of the abutment must incorporate a subdrain as shown in OPSD 3101.150.

## 16 SEISMIC CONSIDERATIONS

In accordance with the CHBDC 2019, the selection of the seismic site classification is based on the averaged soil conditions encountered in the upper 30 m of the stratigraphy. In general, the subsurface conditions at the site consist of a pavement structure and sand fill overlying native layers of clayey silt and silty clay above a deposit of sandy silt to silty sand. The sandy silt to silty sand is underlain by a lower silty clay layer which is in turn underlain by a deposit of silty clay till. This would correspond to a Seismic Site Class D in accordance with Table 4.1, Clause 4.4.3.2 of the CHBDC. The peak ground acceleration, PGA, for a 2% in 50 year probability of exceedance at this site is 0.075 g as per the National Building Code of Canada (NBCC). Since this site is classified as Class D the factored PGA for a 2% in 50 year probability of exceedance at this site is 0.097 g.

In accordance with Clause 6.14.7 of the CHBDC 2019, retaining structures should be designed using active (KAE) and passive (KPE) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 16.1 may be used:

**Table 16.1 – Earth Pressure Coefficients for Earthquake Loading**

| Condition | Earth Pressure Coefficient (K) | |
|---|---|---|
| | OPSS Granular A or Granular B Type II $\phi = 35°$, $\gamma = 22.8$ kN/m$^3$ | OPSS Granular B Type I $\phi = 32°$, $\gamma = 21.2$ kN/m$^3$ |
| Active ($K_{AE}$)* | 0.31 | 0.35 |
| Passive ($K_{PE}$) | 3.6 | 3.1 |
| At Rest ($K_{OE}$)** | 0.55 | 0.6 |

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods

Based on the subsurface conditions, liquefaction is not considered to be a concern at this site.

## 17 CORROSION AND SULPHATE ATTACK POTENTIAL

The results of the corrosivity and sulphate analytical tests conducted on the native soils during the current investigation indicates the following conditions at the locations tested:

- The potential for sulphate attack on concrete foundations from the surrounding fill and native soils is considered to be negligible due to the low concentration of sulphate and chloride in the samples tested.
- The potential for soil corrosion on metal is considered to be high given the low resistivity values measured on the tested samples.
- Appropriate protection measures commensurate with the above are recommended if metal structural elements are used. The effects of road de-icing salts should be also considered.

## 18 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- Driven piles may achieve refusal within the hard/dense zones at varying elevations.
- Although not encountered in the boreholes drilled, glacial deposits inherently contain cobbles and boulders, which may affect installation of piles/caissons. The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions and extend the piles/caissons to competent foundation level.
- Demolition of existing footings has potential to disturb the subgrade. Consideration could be given to leaving the footings in place provided they do not interfere with the abutment construction and new foundations.
- Caisson installation will extend through cohesionless soils below the groundwater table. Therefore, temporary steel liners and stabilization using a synthetic slurry will be required to support the caisson sidewalls and to provide seepage cut-off where required. If accumulated water in the caisson hole cannot be removed, consideration should then be given to using the tremie technique to place concrete inside the caisson hole.
- Vibration monitoring should be carried out during pile driving (if piles selected) and during existing structure demolition to limit potential impacts on existing facilities and residents, and conditions carefully monitored for signs of disturbance. A preconstruction condition survey of

existing structures and utilities should be carried out prior to commencement of pile installation and structure demolition to confirm that the vibration levels are within tolerable limits.

## 19 CLOSURE

Engineering analysis and preparation of the report were carried out by Mr. Geoff Lay, P.Eng..

The report was reviewed by Mr. Jason Lee, P.Eng., and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.


Geoff Lay, P.Eng.
Geotechnical Engineer


Jason Lee, P.Eng.,
Principal/Senior Geotechnical Engineer


P.K. Chatterji, P.Eng.
Review Principal, Designated MTO Contact

THURBER

# Appendix A
# Record of Borehole Sheets, Laboratory Test Results for Present Site Investigation and Analytical Laboratory Test Results

# SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

| CLASSIFICATION | PARTICLE SIZE | VISUAL IDENTIFICATION |
|---|---|---|
| Boulders | Greater than 200mm | same |
| Cobbles | 75 to 200mm | same |
| Gravel | 4.75 to 75mm | 5 to 75mm |
| Sand | 0.075 to 4.75mm | Not visible particles to 5mm |
| Silt | 0.002 to 0.075mm | Non-plastic particles, not visible to the naked eye |
| Clay | Less than 0.002mm | Plastic particles, not visible to the naked eye |

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

| TERMINOLOGY | PROPORTION |
|---|---|
| Trace or Occasional | Less than 10% |
| Some | 10 to 20% |
| Adjective (e.g. silty or sandy) | 20 to 35% |
| And (e.g. sand and gravel) | 35 to 50% |

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

| DESCRIPTIVE TERM | UNDRAINED SHEAR STRENGTH (kPa) | APPROXIMATE SPT(1) 'N' VALUE |
|---|---|---|
| Very Soft | 12 or less | Less than 2 |
| Soft | 12 to 25 | 2 to 4 |
| Firm | 25 to 50 | 4 to 8 |
| Stiff | 50 to 100 | 8 to 15 |
| Very Stiff | 100 to 200 | 15 to 30 |
| Hard | Greater than 200 | Greater than 30 |

NOTE: Hierarchy of Soil Strength Prediction

1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT value
5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

| DESCRIPTIVE TERM | SPT "N" VALUE |
|---|---|
| Very Loose | Less than 4 |
| Loose | 4 to 10 |
| Compact | 10 to 30 |
| Dense | 30 to 50 |
| Very Dense | Greater than 50 |

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

| | | | |
|---|---|---|---|
| SS Split Spoon Sample | WS Wash Sample | AS Auger (Grab) Sample | |
| TW Thin Wall Shelby Tube Sample | | TP Thin Wall Piston Sample | |
| PH Sampler Advanced by Hydraulic Pressure | | PM Sampler Advanced by Manual Pressure | |
| WH Sampler Advanced by Self Static Weight | | RC Rock Core | SC Soil Core |

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

Water Level

$C_{pen}$ Shear Strength Determination by Pocket Penetrometer

(1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

<u>UNIFIED SOILS CLASSIFICATION</u>

| MAJOR DIVISIONS | | GROUP SYMBOL | TYPICAL DESCRIPTION |
|---|---|---|---|
| COARSE GRAINED SOILS | GRAVEL AND GRAVELLY SOILS | GW | Well-graded gravels or gravel-sand mixtures, little or no fines. |
| | | GP | Poorly-graded gravels or gravel-sand mixtures, little or no fines. |
| | | GM | Silty gravels, gravel-sand-silt mixtures. |
| | | GC | Clayey gravels, gravel-sand-clay mixtures. |
| | SAND AND SANDY SOILS | SW | Well-graded sands or gravelly sands, little or no fines. |
| | | SP | Poorly-graded sands or gravelly sands, little or no fines. |
| | | SM | Silty sands, sand-silt mixtures. |
| | | SC | Clayey sands, sand-clay mixtures. |
| FINE GRAINED SOILS | SILTS AND CLAYS $W_L < 50\%$ | ML | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity. |
| | | CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$). |
| | | CI | Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$). |
| | | OL | Organic silts and organic silty-clays of low plasticity. |
| | SILTS AND CLAYS $W_L > 50\%$ | MH | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts. |
| | | CH | Inorganic clays of high plasticity, fat clays. |
| | | OH | Organic clays of medium to high plasticity, organic silts. |
| HIGHLY ORGANIC SOILS | | Pt | Peat and other highly organic soils. |
| CLAY SHALE | | | |
| SANDSTONE | | | |
| SILTSTONE | | | |
| CLAYSTONE | | | |
| COAL | | | |

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# RECORD OF BOREHOLE No BH20-01



**METRIC**

GWP# 408-88-00 | LOCATION , MTM NAD 83 Zone 10: N 4 813 653.3 E 226 144.0 | ORIGINATED BY MC

DIST ______ HWY 7 | BOREHOLE TYPE Hollow Stem Augers/Tricone | COMPILED BY AN

DATUM Geodetic | DATE 2020.08.17 - 2020.08.19 | LATITUDE 43.458660 | LONGITUDE -80.471975 | CHECKED BY GRL

| ELEV / DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | WATER CONTENT (%) | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|
| 327.5 | GROUND SURFACE | | | | | | | | |
| 0.0 | **ASPHALT:** (200mm) | | | | | | | | |
| 0.2 | **SAND** and **GRAVEL** Brown Dry (FILL) | | | | | | | | |
| 326.7 / 0.8 | **SAND**, some silt, some gravel Compact Brown Dry (FILL) | 1 | SS | 28 | | | | | |
| | | 2 | SS | 12 | | | | | |
| 325.3 / 2.2 | **SAND**, trace silt Very Loose to Loose Brown Dry (FILL) | 3 | SS | 3 | | | | | |
| | | 4 | SS | 8 | | 0 | 89 | 11 (SI+CL) | |
| 323.4 / 4.1 | **SAND**, trace silt Compact Brown Wet | 5 | SS | 27 | | | | | |
| | | 6 | SS | 17 | | | | | |
| 320.3 / 7.2 | Clayey **SILT**, trace sand, trace gravel Stiff Brown Wet | 7 | SS | 9 | | 1 | 7 | 78 | 14 |
| 318.8 / 8.7 | Silty **CLAY**, trace sand Very Stiff to Hard Grey Wet | 8 | SS | 31 | | | | | |

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (327–318); 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; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH20-01



**METRIC**

GWP# 408-88-00 LOCATION , MTM NAD 83 Zone 10: N 4 813 653.3 E 226 144.0 ORIGINATED BY MC

DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers/Tricone COMPILED BY AN

DATUM Geodetic DATE 2020.08.17 - 2020.08.19 LATITUDE 43.458660 LONGITUDE -80.471975 CHECKED BY GRL

Continued From Previous Page

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| | | | | | 317 | |
| | | 9 | SS | 24 | | 0 0 30 70 |
| | | | | | 316 | |
| | | 10 | SS | 31 | 315 | |
| 314.2 / 13.3 | Silty **SAND** to Sandy **SILT**, trace clay<br>Very Dense to Dense<br>Grey<br>Wet | | | | 314 | |
| | | 11 | SS | 72 | 313 | |
| | | 12 | SS | 85 | 312 | |
| | | | | | 311 | |
| | | 13 | SS | 88 | | 0 28 66 6 |
| | | | | | 310 | |
| | | 14 | SS | 45 | 309 | |
| 308.1 / 19.4 | Silty **CLAY**, trace sand<br>Hard<br>Grey<br>Wet | | | | 308 | |

Continued Next Page

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

20 15 5 10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH20-01



**METRIC**

GWP# 408-88-00 LOCATION , MTM NAD 83 Zone 10: N 4 813 653.3 E 226 144.0 ORIGINATED BY MC

DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers/Tricone COMPILED BY AN

DATUM Geodetic DATE 2020.08.17 - 2020.08.19 LATITUDE 43.458660 LONGITUDE -80.471975 CHECKED BY GRL

SOIL PROFILE: ELEV DEPTH | DESCRIPTION | STRAT PLOT

SAMPLES: NUMBER | TYPE | "N" VALUES

GROUND WATER CONDITIONS | ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

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

Continued From Previous Page

| Sample NUMBER | TYPE | "N" VALUES | GR | SA | SI | CL |
|---|---|---|---|---|---|---|
| 15 | SS | 39 | | | | |
| 16 | SS | 37 | | | | |
| 17 | SS | 32 | | | | |
| 18 | SS | 30 | 0 | 4 | 36 | 60 |

Elevation scale: 307, 306, 305, 304, 303, 302, 301, 300, 299, 298

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH20-01



**METRIC**

GWP# 408-88-00 | LOCATION , MTM NAD 83 Zone 10: N 4 813 653.3 E 226 144.0 | ORIGINATED BY MC

DIST ____ HWY 7 | BOREHOLE TYPE Hollow Stem Augers/Tricone | COMPILED BY AN

DATUM Geodetic | DATE 2020.08.17 - 2020.08.19 | LATITUDE 43.458660 | LONGITUDE -80.471975 | CHECKED BY GRL

| ELEV / DEPTH | DESCRIPTION (SOIL PROFILE) | SAMPLE NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| | Continued From Previous Page | | | | | |
| | | | | | 297 | |
| | | | | | 296 | |
| | Very Stiff | 19 | SS | 23 | | |
| | | | | | 295 | |
| | | | | | 294 | |
| 293.7 / 33.8 | Silty **CLAY**, sandy, trace gravel<br>Hard<br>Grey<br>Wet<br>(TILL) | | | | | |
| | | | | | 293 | |
| | | 20 | SS | 100/ 0.275 | | |
| | | | | | 292 | |
| | | | | | 291 | |
| | | 21 | SS | 76/ 0.250 | | 3 31 51 15 |
| | | | | | 290 | |
| 289.2 / 38.3 | | 22 | SS | 105/ 0.175 | | |

END OF BOREHOLE AT 38.3m.
Well installation consists of 50mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen.

WATER LEVEL READINGS

| DATE | DEPTH(m) | ELEV.(m) |
|---|---|---|
| 2020.08.24 | 5.5 | 322.0 |

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT $w_P$, NATURAL MOISTURE CONTENT $w$, LIQUID LIMIT $w_L$; WATER CONTENT (%) 20 40 60; UNIT WEIGHT $\gamma$ kN/m$^3$; STRAT PLOT; GROUND WATER CONDITIONS.

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

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# RECORD OF BOREHOLE No BH20-02



**METRIC**

GWP# 408-88-00 | LOCATION , MTM NAD 83 Zone 10: N 4 813 695.8 E 226 245.9 | ORIGINATED BY MC

DIST ____ HWY 7 | BOREHOLE TYPE Hollow Stem Augers/Tricone | COMPILED BY AN

DATUM Geodetic | DATE 2020.08.20 - 2020.08.21 | LATITUDE 43.459054 | LONGITUDE -80.470721 | CHECKED BY GRL

| ELEV / DEPTH | DESCRIPTION (SOIL PROFILE) | SAMPLE NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE |
|---|---|---|---|---|---|---|
| 325.0 | GROUND SURFACE | | | | | |
| 0.0 | **ASPHALT:** (163mm) | | | | | |
| 0.2 | **SAND** and **GRAVEL** Brown Dry (FILL) | | | | | |
| 324.3 / 0.7 | **SAND**, gravelly to some gravel Loose to Very Loose Brown Dry (FILL) | 1 | SS | 7 | | 324 |
| | | 2 | SS | 1 | | 323 |
| 322.7 / 2.3 | **SAND**, trace silt Loose to Compact Brown Dry | 3 | SS | 6 | ∇ | 322 |
| | | 4 | SS | 12 | | 321 |
| | Wet | 5 | SS | 13 | | 320 |
| | | 6 | SS | 14 | | 319, 318 |
| 317.8 / 7.2 | Silty **CLAY**, trace sand Very Stiff to Hard Grey Wet | 7 | SS | 25 | | 317 |
| | | 8 | SS | 34 | | 316, 315 |

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

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

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH20-02



**METRIC**

GWP# 408-88-00 | LOCATION , MTM NAD 83 Zone 10: N 4 813 695.8 E 226 245.9 | ORIGINATED BY MC
DIST | HWY 7 | BOREHOLE TYPE Hollow Stem Augers/Tricone | COMPILED BY AN
DATUM Geodetic | DATE 2020.08.20 - 2020.08.21 | LATITUDE 43.459054 | LONGITUDE -80.470721 | CHECKED BY GRL

Continued From Previous Page

| ELEV / DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|
| | | 9 | SS | 25 | 314 | | | | |
| | | | | | 313 | | | | |
| | | 10 | SS | 31 | | 0 | 0 | 34 | 66 |
| | | | | | 312 | | | | |
| 310.9 / 14.1 | **SAND** Dense Brown Wet | 11 | SS | 34 | 311 | | | | |
| 310.2 / 14.8 | | | | | 310 | | | | |
| | | 12 | SS | 33 | 309 | | | | |
| 308.7 / 16.3 | **SAND** Very Dense Brown Wet | | | | | | | | |
| | | 13 | SS | 71 | 308 | | | | |
| 307.2 / 17.8 | Silty **CLAY**, trace sand Hard Grey Wet | | | | 307 | | | | |
| | | 14 | SS | 50 | 306 | 0 | 1 | 43 | 56 |
| | | | | | 305 | | | | |

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT $w_P$, NATURAL MOISTURE CONTENT $w$, LIQUID LIMIT $w_L$; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m$^3$; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH20-02



**METRIC**

GWP# 408-88-00 | LOCATION , MTM NAD 83 Zone 10: N 4 813 695.8 E 226 245.9 | ORIGINATED BY MC

DIST HWY 7 | BOREHOLE TYPE Hollow Stem Augers/Tricone | COMPILED BY AN

DATUM Geodetic | DATE 2020.08.20 - 2020.08.21 | LATITUDE 43.459054 | LONGITUDE -80.470721 | CHECKED BY GRL

| 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 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 | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Continued From Previous Page | | | | | | | | | | |
| | | | 15 | SS | 103 | | | | | | |
| | | | | | | | 304 | | | | |
| | | | 16 | SS | 57 | | | | ○ | | |
| | | | | | | | 303 | | | | |
| | | | | | | | 302 | | | | |
| | | | | | | | 301 | | | | |
| | | | 17 | SS | 44 | | | | ├○──┤ | | 0 2 37 61 |
| | | | | | | | 300 | | | | |
| | | | | | | | 299 | | | | |
| | | | | | | | 298 | | | | |
| | Very Stiff | | 18 | SS | 28 | | | | ○ | | |
| | | | | | | | 297 | | | | |
| | | | | | | | 296 | | | | |
| | | | | | | | 295 | | | | |

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

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# RECORD OF BOREHOLE No BH20-02



**METRIC**

GWP# 408-88-00
LOCATION , MTM NAD 83 Zone 10: N 4 813 695.8 E 226 245.9
ORIGINATED BY MC

DIST HWY 7
BOREHOLE TYPE Hollow Stem Augers/Tricone
COMPILED BY AN

DATUM Geodetic
DATE 2020.08.20 - 2020.08.21
LATITUDE 43.459054
LONGITUDE -80.470721
CHECKED BY GRL

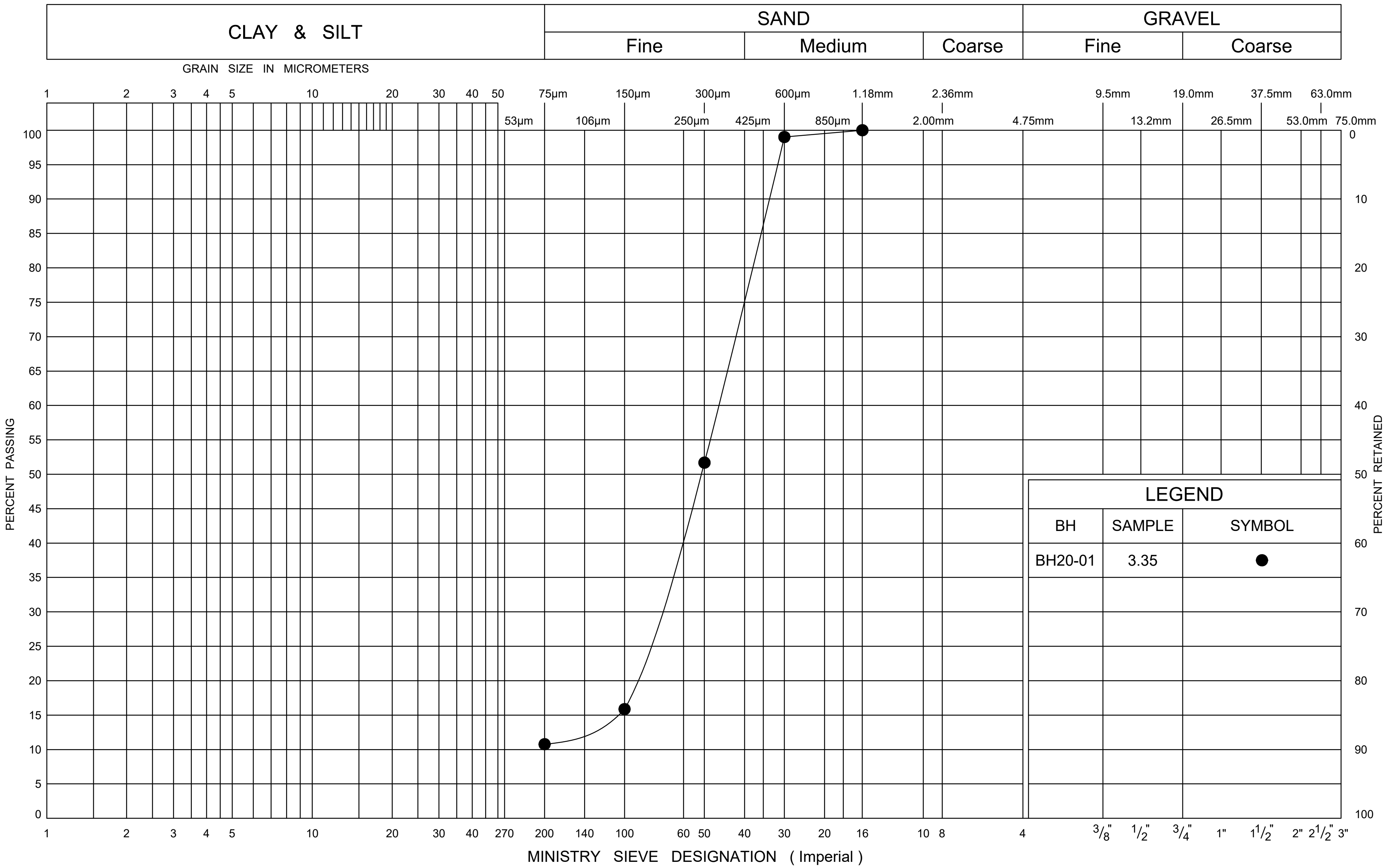
| ELEV / DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | WATER CONTENT (%) | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|
| | Continued From Previous Page | | | | | | | | | |
| | | 19 | SS | 24 | 294 | | | | | |
| 292.7 / 32.3 | **SAND**, with gravel, trace silt<br>Very Dense<br>Grey<br>Wet | | | | 293 | | | | | |
| | | 20 | SS | 69 | 292 / 291 | | | | | |
| 290.4 / 34.6 | Silty **CLAY**, with sand, gravelly<br>Hard<br>Grey<br>Wet<br>(TILL) | | | | 290 | | | | | |
| | | 21 | SS | 101/ 0.275 | | | 21 | 36 | 28 | 15 |
| | | 22 | SS | 103/0.225 | 289 / 288 | | | | | |
| | | 23 | SS | 104/ 0.250 | 287 | | | | | |
| 286.6 / 38.4 | END OF BOREHOLE AT 38.35m.<br>WATER LEVEL AT 2.3m.<br>BOREHOLE BACKFILLED WITH BENTONITE CUTTINGS AND ASPHALT COLD PATCH TO SURFACE. | | | | | | | | | |

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT $w_P$, NATURAL MOISTURE CONTENT $w$, LIQUID LIMIT $w_L$; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m$^3$; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

78 12 M

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# GRAIN SIZE DISTRIBUTION

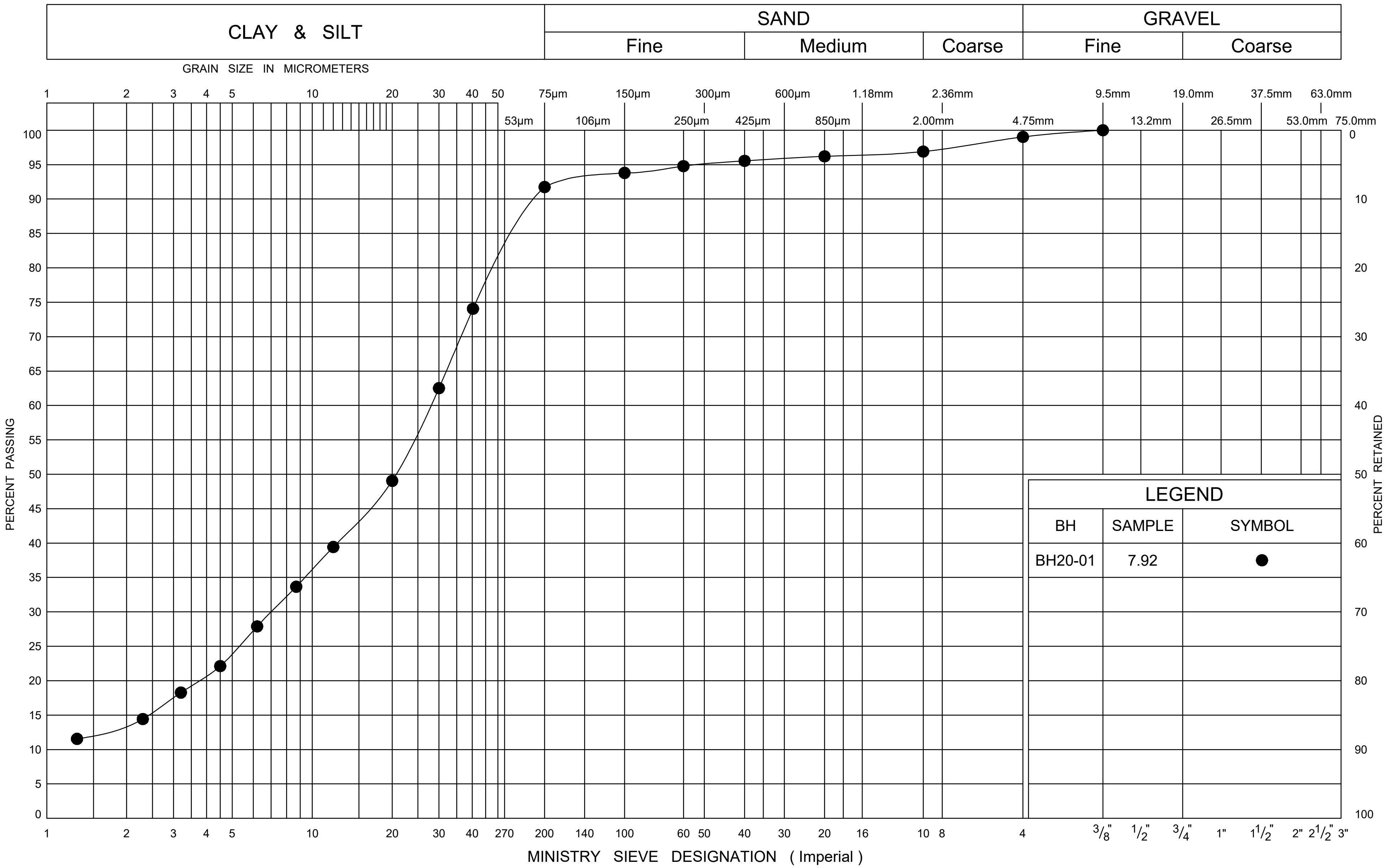
## SAND (FILL)

FIG No A1

W P 408-88-00

ONTARIO MOT GRAIN SIZE 2 MTO-11375(GINTDATA).GPJ ONTARIO MOT.GDT 11/26/20

78 12 M

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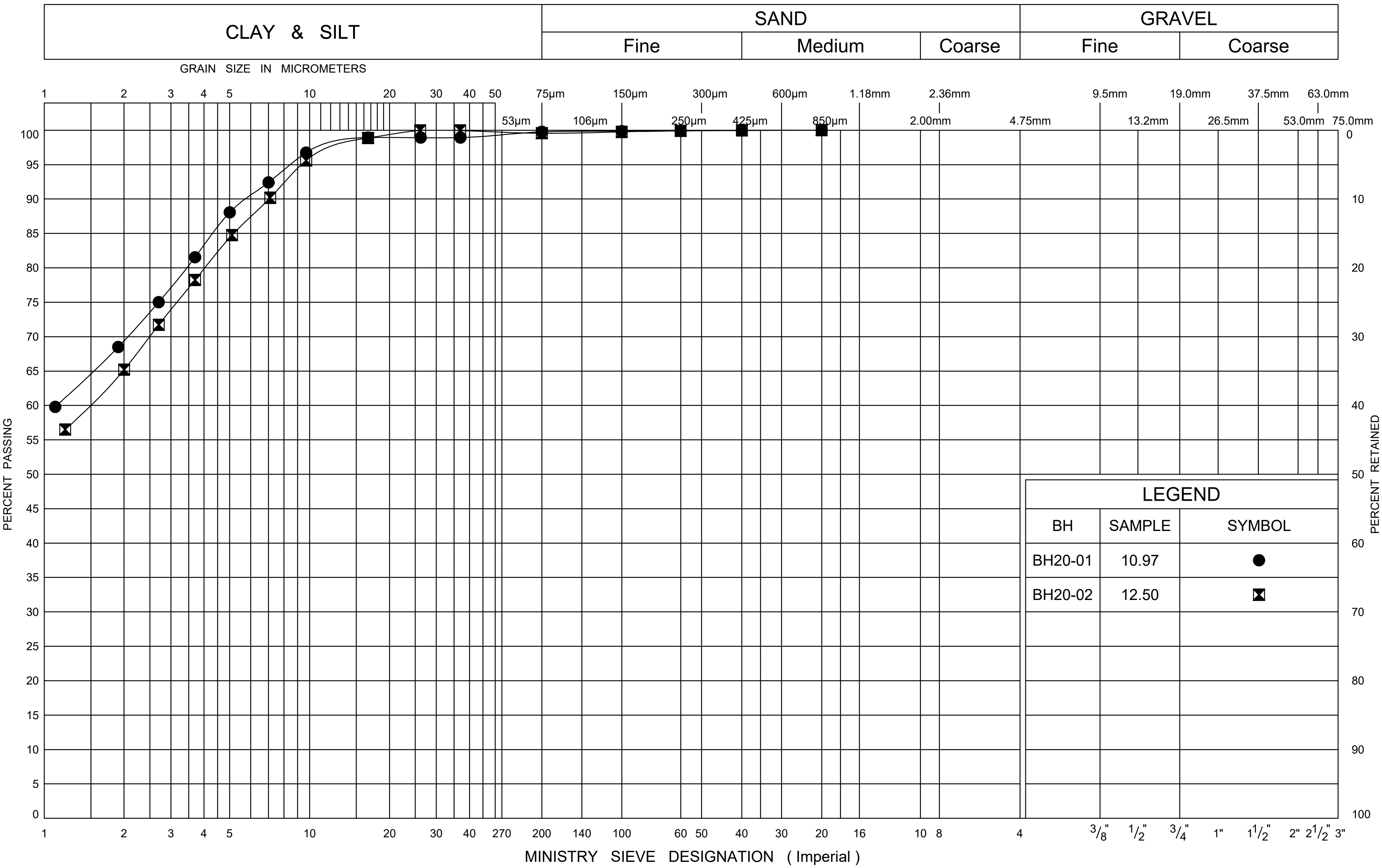
# GRAIN SIZE DISTRIBUTION

Clayey SILT

FIG No A2

W P 408-88-00




| BH | SAMPLE | SYMBOL |
|---|---|---|
| BH20-01 | 10.97 | ● |
| BH20-02 | 12.50 | ⊠ |

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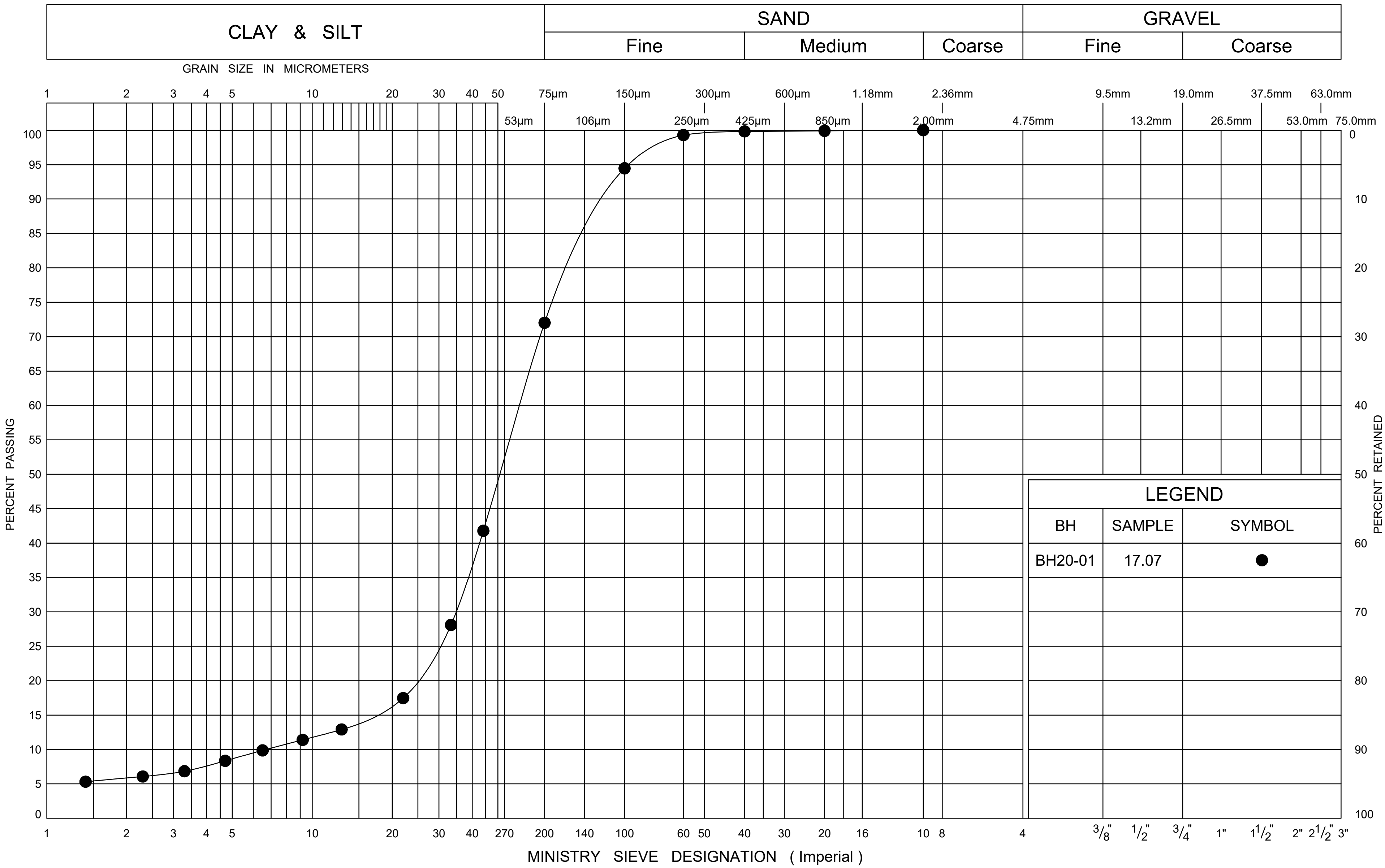
# GRAIN SIZE DISTRIBUTION

## Upper Silty CLAY

FIG No A3

W P 408-88-00



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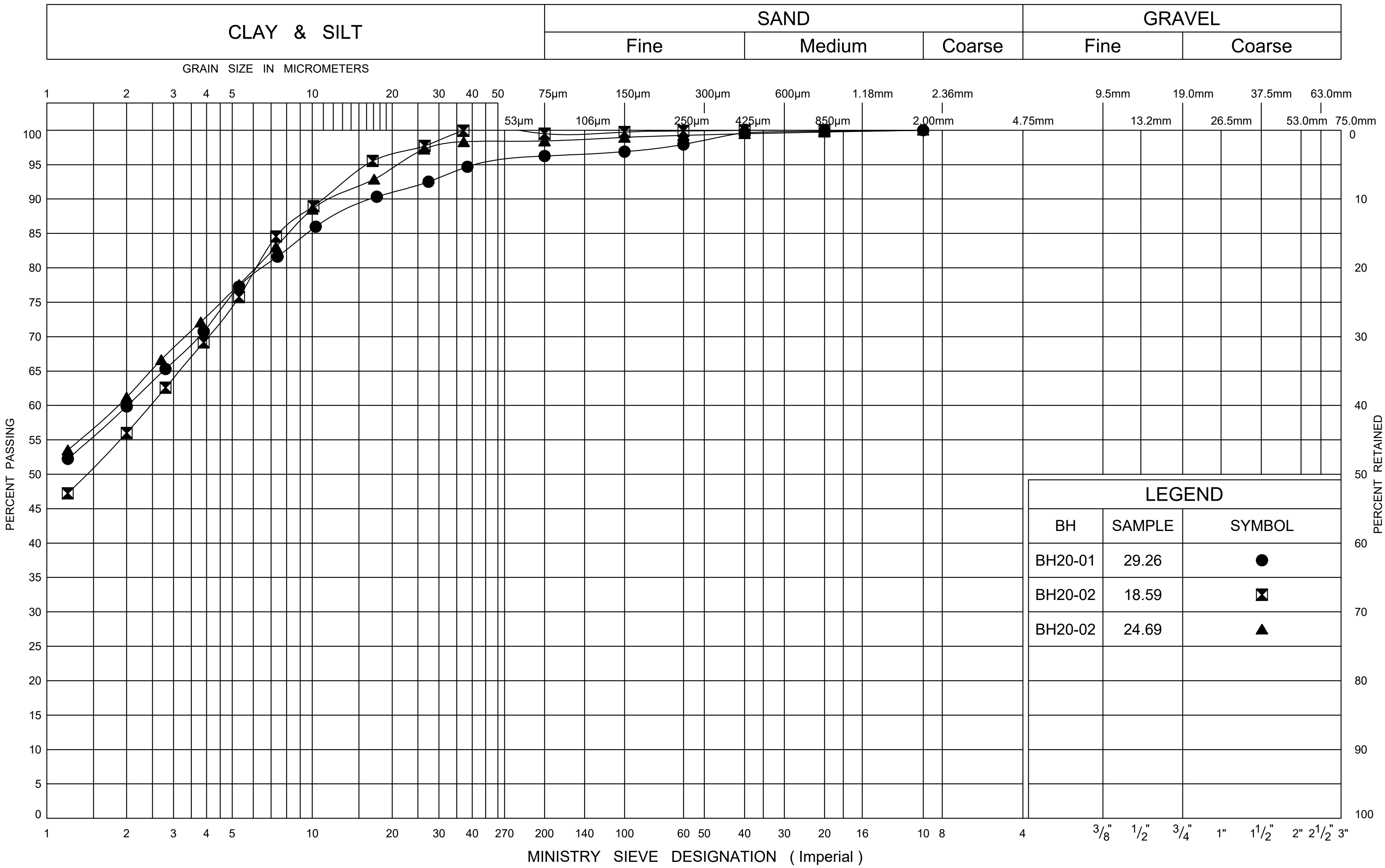
# GRAIN SIZE DISTRIBUTION

Sandy SILT to Silty SAND

FIG No A4

W P 408-88-00




LEGEND

| BH | SAMPLE | SYMBOL |
|---|---|---|
| BH20-01 | 29.26 | ● |
| BH20-02 | 18.59 | ⧗ |
| BH20-02 | 24.69 | ▲ |

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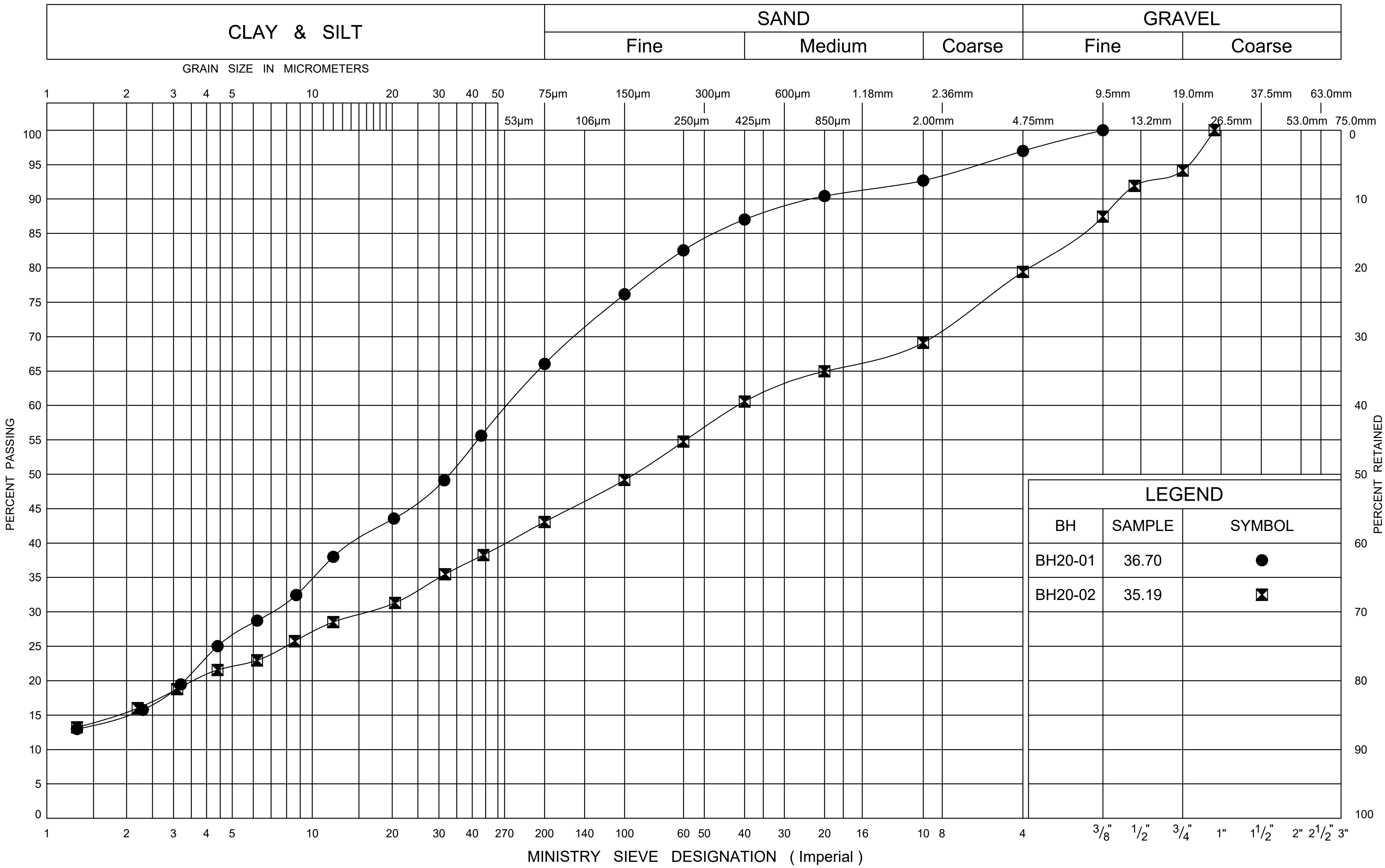
# GRAIN SIZE DISTRIBUTION

Lower Silty CLAY

FIG No A5

W P 408-88-00




| LEGEND | | |
|---|---|---|
| BH | SAMPLE | SYMBOL |
| BH20-01 | 36.70 | ● |
| BH20-02 | 35.19 | ⊠ |

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Ontario

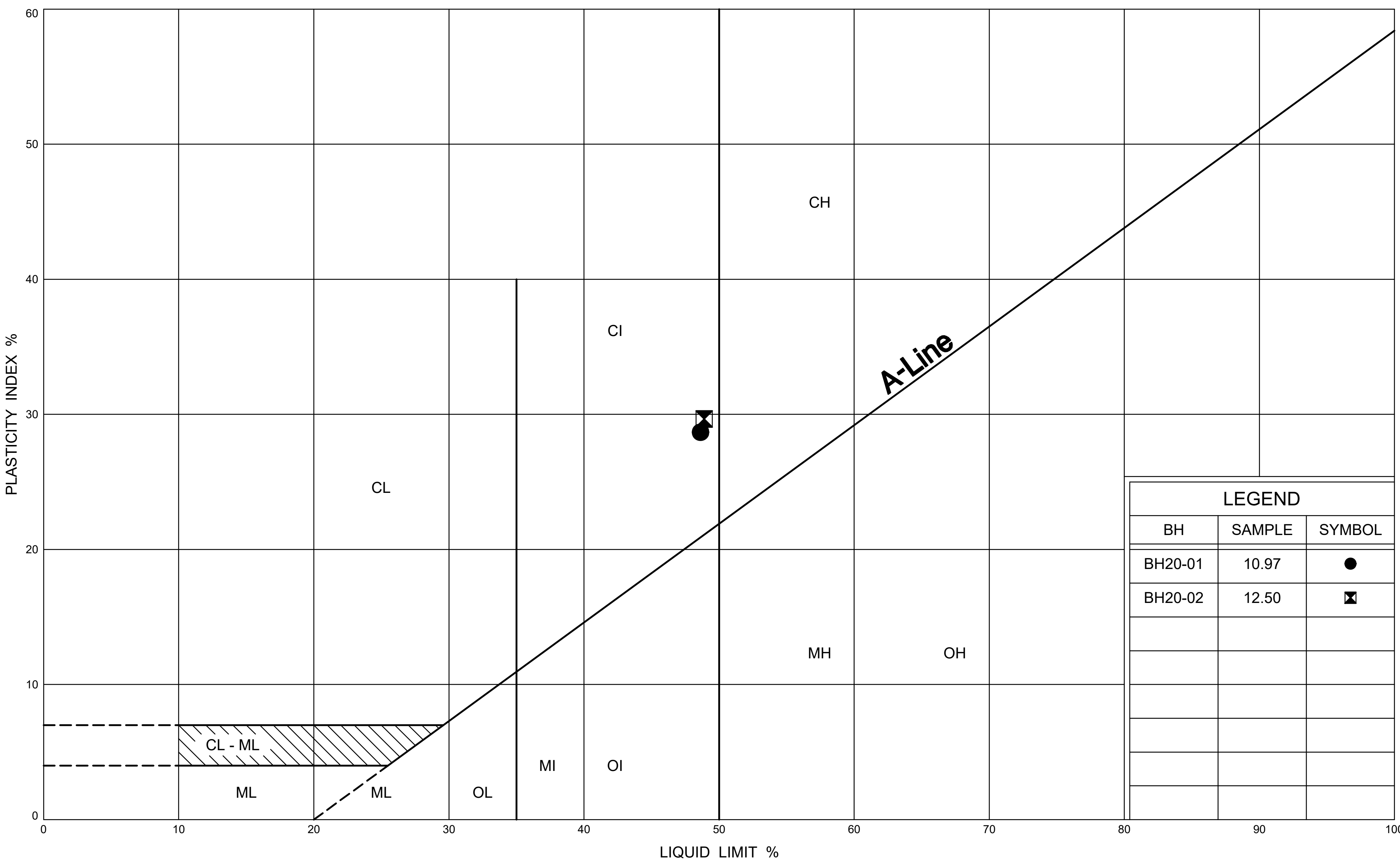
# GRAIN SIZE DISTRIBUTION

## Silty CLAY TILL

FIG No A6

W P 408-88-00

Oct 75, FF - S - 21




Ministry of Transportation

Ontario

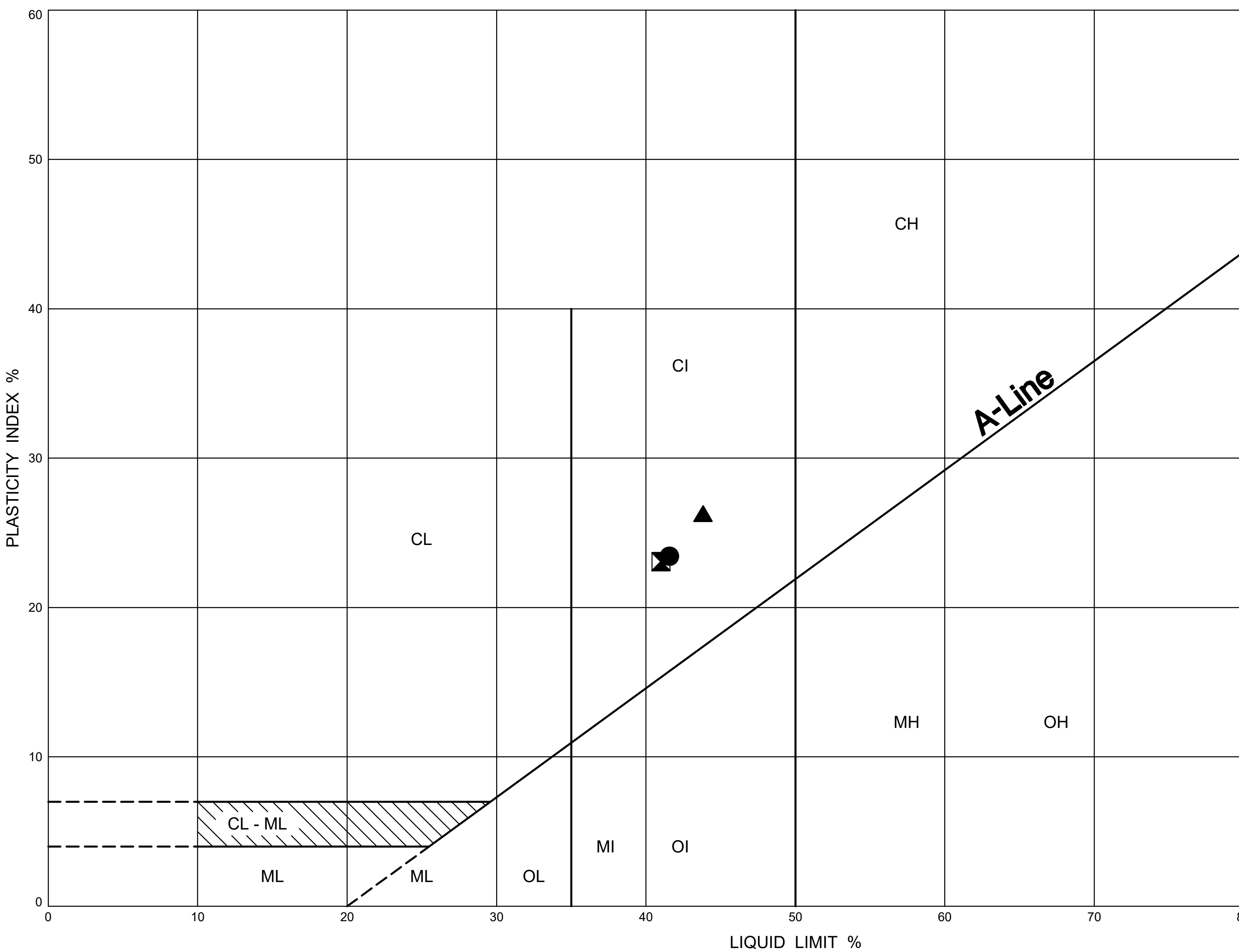
# PLASTICITY CHART

## Upper Silty CLAY

FIG No A7

W P 408-88-00

Oct 75, FF - S - 21




LEGEND

| BH | SAMPLE | SYMBOL |
|---|---|---|
| BH20-01 | 29.26 | ● |
| BH20-02 | 18.59 | ⧗ |
| BH20-02 | 24.69 | ▲ |
| | | |
| | | |
| | | |
| | | |
| | | |

Ministry of Transportation
Ontario

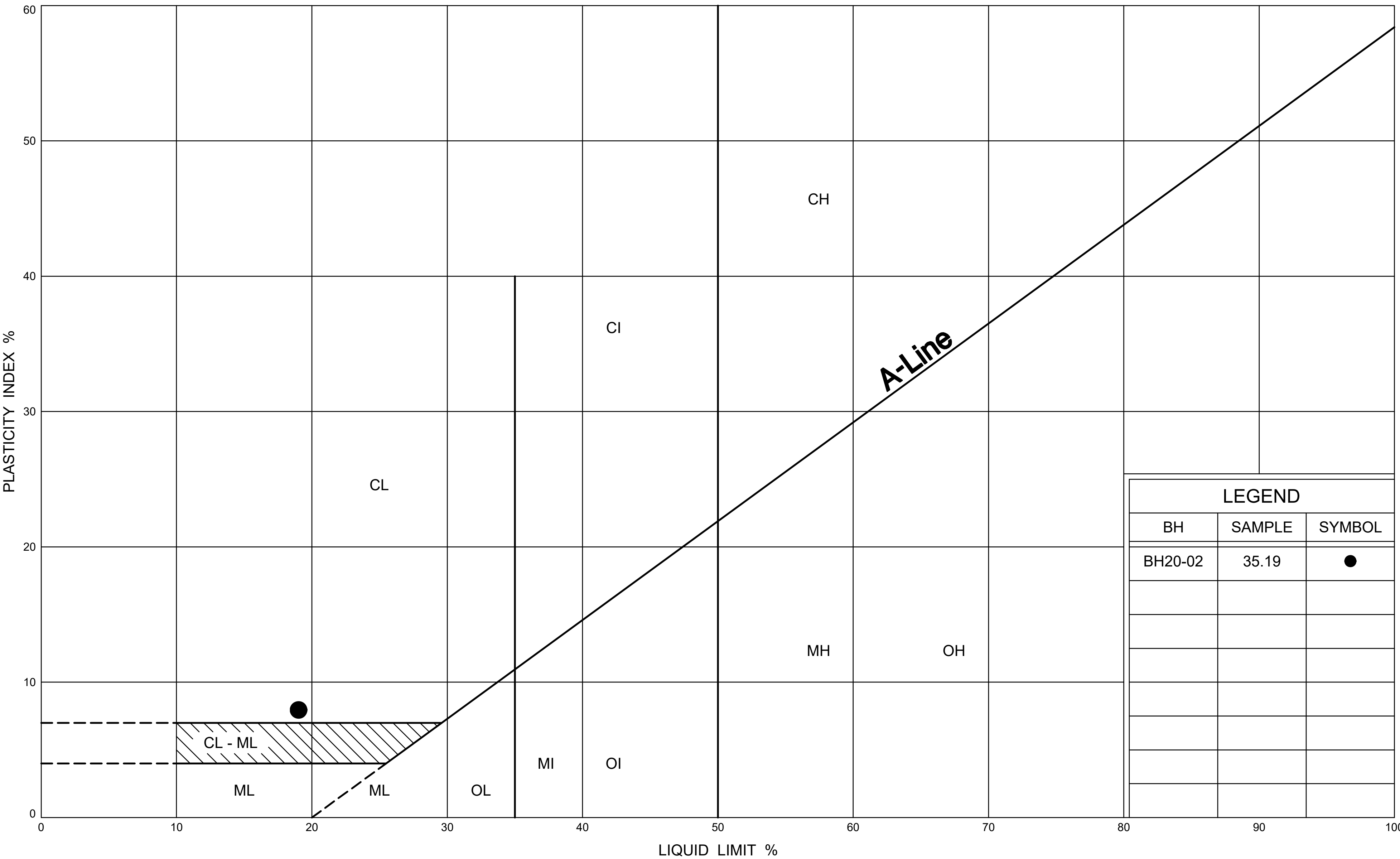
# PLASTICITY CHART
## Lower Silty CLAY

FIG No A8

W P 408-88-00

ONTARIO MOT PLASTICITY CHART MTO-11375(GINTDATA).GPJ ONTARIO MOT.GDT 11/26/20

Oct 75, FF - S - 21




Ministry of Transportation

Ontario

# PLASTICITY CHART

## Silty CLAY TILL

FIG No A9

W P 408-88-00


# FINAL REPORT

## CA14882-AUG20 R1

### 1375 Frederick St.

Prepared for

**Thurber Engineering Ltd.**

# FINAL REPORT

## First Page

| CLIENT DETAILS | | LABORATORY DETAILS | |
|---|---|---|---|
| Client | Thurber Engineering Ltd. | Project Specialist | Jill Campbell, B.Sc.,GISAS |
| | | Laboratory | SGS Canada Inc. |
| Address | 103, 2010 Winston Park Drive<br>Oakville, ON<br>L6H 5R7. Canada | Address | 185 Concession St., Lakefield ON, K0L 2H0 |
| Contact | Geoff Lay | Telephone | 2165 |
| Telephone | 905-829-8666 | Facsimile | 705-652-6365 |
| Facsimile | | Email | jill.campbell@sgs.com |
| Email | glay@thurber.ca | SGS Reference | CA14882-AUG20 |
| Project | 1375 Frederick St. | Received | 08/28/2020 |
| Order Number | | Approved | 09/03/2020 |
| Samples | Soil (2) | Report Number | CA14882-AUG20 R1 |
| | | Date Reported | 09/03/2020 |

COMMENTS

Temperature of Sample upon Receipt:7 degrees C
Cooling Agent Present:YES
Custody Seal Present:YES

Chain of Custody Number:NA

Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.

SIGNATORIES

Jill Campbell, B.Sc.,GISAS

Jill Campbell

## TABLE OF CONTENTS

# FINAL REPORT



**Client:** Thurber Engineering Ltd.

**Project:** 1375 Frederick St.

**Project Manager:** Geoff Lay

**Samplers:** Brett Thomas

PACKAGE: **- Corrosivity Index** (SOIL)

| | | | Sample Number | 5 | 6 |
|---|---|---|---|---|---|
| | | | **Sample Name** | BH20-01 SS#4 | BH20-02 SS#3 |
| | | | **Sample Matrix** | Soil | Soil |
| | | | **Sample Date** | 17/08/2020 | 20/08/2020 |
| **Parameter** | **Units** | **RL** | | **Result** | **Result** |
| **Corrosivity Index** | | | | | |
| Corrosivity Index | none | 1 | | 8 | 13 |
| Soil Redox Potential | mV | - | | 287 | 285 |
| Sulphide | % | 0.04 | | < 0.04 | < 0.04 |
| pH | pH Units | 0.05 | | 9.66 | 9.37 |
| Resistivity (calculated) | ohms.cm | -9999 | | 1830 | 892 |

PACKAGE: **- General Chemistry** (SOIL)

| | | | Sample Number | 5 | 6 |
|---|---|---|---|---|---|
| | | | **Sample Name** | BH20-01 SS#4 | BH20-02 SS#3 |
| | | | **Sample Matrix** | Soil | Soil |
| | | | **Sample Date** | 17/08/2020 | 20/08/2020 |
| **Parameter** | **Units** | **RL** | | **Result** | **Result** |
| **General Chemistry** | | | | | |
| Conductivity | uS/cm | 2 | | 547 | 1120 |

PACKAGE: **- Metals and Inorganics** (SOIL)

| | | | Sample Number | 5 | 6 |
|---|---|---|---|---|---|
| | | | **Sample Name** | BH20-01 SS#4 | BH20-02 SS#3 |
| | | | **Sample Matrix** | Soil | Soil |
| | | | **Sample Date** | 17/08/2020 | 20/08/2020 |
| **Parameter** | **Units** | **RL** | | **Result** | **Result** |
| **Metals and Inorganics** | | | | | |
| Moisture Content | % | 0.1 | | 3.8 | 4.4 |
| Sulphate | µg/g | 0.4 | | 8.3 | 21 |

# FINAL REPORT

CA14882-AUG20 R1

**Client:** Thurber Engineering Ltd.

**Project:** 1375 Frederick St.

**Project Manager:** Geoff Lay

**Samplers:** Brett Thomas

PACKAGE: **- Other (ORP)** (SOIL)

| | | | Sample Number | 5 | 6 |
|---|---|---|---|---|---|
| | | | Sample Name | BH20-01 SS#4 | BH20-02 SS#3 |
| | | | Sample Matrix | Soil | Soil |
| | | | Sample Date | 17/08/2020 | 20/08/2020 |
| **Parameter** | **Units** | **RL** | | **Result** | **Result** |
| **Other (ORP)** | | | | | |
| Chloride | µg/g | 0.4 | | 210 | 750 |

## QC SUMMARY

### Anions by IC

Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-[ENV]IC-LAK-AN-001

| Parameter | QC batch Reference | Units | RL | Method Blank | Duplicate | | LCS/Spike Blank | | | Matrix Spike / Ref. | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | RPD | AC (%) | Spike Recovery (%) | Recovery Limits (%) Low | Recovery Limits (%) High | Spike Recovery (%) | Recovery Limits (%) Low | Recovery Limits (%) High |
| Chloride | DIO0461-AUG20 | µg/g | 0.4 | <0.4 | 2 | 20 | 96 | 80 | 120 | 103 | 75 | 125 |
| Sulphate | DIO0461-AUG20 | µg/g | 0.4 | <0.4 | 8 | 20 | 98 | 80 | 120 | 95 | 75 | 125 |

### Carbon/Sulphur

Method: ASTM E1915-07A | Internal ref.: ME-CA-[ENV]ARD-LAK-AN-020

| Parameter | QC batch Reference | Units | RL | Method Blank | Duplicate | | LCS/Spike Blank | | | Matrix Spike / Ref. | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | RPD | AC (%) | Spike Recovery (%) | Recovery Limits (%) Low | Recovery Limits (%) High | Spike Recovery (%) | Recovery Limits (%) Low | Recovery Limits (%) High |
| Sulphide | ECS0001-SEP20 | % | 0.04 | < 0.04 | ND | 20 | 100 | 80 | 120 | | | |

### Conductivity

Method: SM 2510 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-006

| Parameter | QC batch Reference | Units | RL | Method Blank | Duplicate | | LCS/Spike Blank | | | Matrix Spike / Ref. | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | RPD | AC (%) | Spike Recovery (%) | Recovery Limits (%) Low | Recovery Limits (%) High | Spike Recovery (%) | Recovery Limits (%) Low | Recovery Limits (%) High |
| Conductivity | EWL0414-AUG20 | uS/cm | 2 | < 0.002 | 1 | 20 | 99 | 90 | 110 | NA | | |

## QC SUMMARY

### pH

### Method: SM 4500 I Internal ref.: ME-CA-[ENV]EWL-LAK-AN-001

| Parameter | QC batch Reference | Units | RL | Method Blank | Duplicate | | LCS/Spike Blank | | | Matrix Spike / Ref. | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | RPD | AC (%) | Spike Recovery (%) | Recovery Limits (%) | | Spike Recovery (%) | Recovery Limits (%) | |
| | | | | | | | | Low | High | | Low | High |
| pH | EWL0414-AUG20 | pH Units | 0.05 | NA | 1 | | 100 | | | NA | | |

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate laboratory accuracy with sample matrix effects.

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

**Multielement Scan Qualifier**: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

**Duplicate Qualifier**: for duplicates as the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

**Matrix Spike Qualifier**: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

## LEGEND

### FOOTNOTES

**NSS** Insufficient sample for analysis.
**RL** Reporting Limit.
↑ Reporting limit raised.
↓ Reporting limit lowered.
**NA** The sample was not analysed for this analyte
**ND** Non Detect

Samples analysed as received. Solid samples expressed on a dry weight basis. “Temperature Upon Receipt” is representative of the whole shipment and may not reflect the temperature of individual samples.

Analysis conducted on samples submitted pursuant to or as part of Reg. 153/04, are in accordance to the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act” published by the Ministry and dated March 9, 2004 as amended.



-- End of Analytical Report --

**SGS** Environment, Health & Safety

# Request for Laboratory Services and CHAIN OF CUSTODY

- Lakefield: 185 Concession St., Lakefield, ON K0L 2H0 Phone: 705-652-2000 Fax: 705-652-6365 Web: www.sgs.com/environment
- London: 657 Consortium Court, London, ON, N6E 2S8 Phone: 519-672-4500 Toll Free: 877-848-8060 Fax: 519-672-0361

No:

Page 1 of 1

**Laboratory Information Section - Lab use only**

Received By: Scott R
Received Date: 08/28/2020 (mm/dd/yy)
Received Time: 16:30 (hr : min)

Received By (signature): [signature]
Custody Seal Present: Yes ☑ No ☐
Custody Seal Intact: Yes ☑ No ☐

Cooling Agent Present: Yes ☑ No ☐ Type: Ice
Temperature Upon Receipt (°C): 12, 2, 2°C

LAB LIMS #: 14882

| REPORT INFORMATION | INVOICE INFORMATION |
|---|---|
| Company: Thurber Engineering Ltd. | ☑ (same as Report Information) |
| Contact: Geoff Lay | Company: |
| Address: 2010 Winston Park Drive #103 Oakville, ON L6H 5R7 | Contact: |
| | Address: |
| Phone: (905) 829-8666 | |
| Fax: | Phone: |
| Email: glay@thurber.ca | Email: |

Quotation #:
P.O. #:
Project #: 11375
Site Location/ID: Frederick St

**TURNAROUND TIME (TAT) REQUIRED**

☑ Regular TAT (5-7days)

TAT's are quoted in business days (exclude statutory holidays & weekends). Samples received after 6pm or on weekends: TAT begins next business day

RUSH TAT (Additional Charges May Apply): ☐ 1 Day ☐ 2 Days ☐ 3 Days ☐ 4 Days

PLEASE CONFIRM RUSH FEASIBILITY WITH SGS REPRESENTATIVE PRIOR TO SUBMISSION

Specify Due Date:

NOTE: DRINKING (POTABLE) WATER SAMPLES FOR HUMAN CONSUMPTION MUST BE SUBMITTED WITH SGS DRINKING WATER CHAIN OF CUSTODY

**REGULATIONS**

Regulation 153/04:
☐ Table 1 ☑ Res/Park Soil Texture:
☑ Table 2 ☐ Ind/Com ☑ Coarse
☐ Table 3 ☐ Agri/Other ☐ Medium/Fine
☐ Table ____

Other Regulations:
☐ Reg 347/558 (3 Day min TAT)
☐ PWQO ☐ MMER
☐ CCME ☐ Other
☐ MISA

Sewer By-Law:
☐ Sanitary
☐ Storm
Municipality:

RECORD OF SITE CONDITION (RSC) ☐ YES ☐ NO

**ANALYSIS REQUESTED**

| # | SAMPLE IDENTIFICATION | DATE SAMPLED | TIME SAMPLED | # OF BOTTLES | MATRIX | Field Filtered (Y/N) | Metals & Inorganics | Full Metals Suite | ICP Metals only | PAHs only | SVOCs | PCBs | F1-F4 + BTEX | F1-F4 only | VOCs | BTEX only | Pesticides | Corrosivity | Other | | | | | Sewer Use | Water Characterization Pkg | TCLP | COMMENTS: |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | BH20-01 SS #4 | Aug.17/2020 | 9 AM | 2 | Soil | | | | | | | | | | | | | ✓ | | | | | | | | | |
| 2 | BH20-02 SS #3 | Aug. 20/2020 | 9 AM | 2 | Soil | | | | | | | | | | | | | ✓ | | | | | | | | | |
| 3 | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 4 | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 5 | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 6 | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | | | | | | | | | | | | | | | | 4day |
| 8 | | | | | | | | | | | | | | | | | | | | | | | | | | | Soil |
| 9 | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 11 | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 12 | | | | | | | | | | | | | | | | | | | | | | | | | | | |

Observations/Comments/Special Instructions

Sampled By (NAME): Brett Thomas | Signature: [signature] | Date: 08 / 28 / 2020 (mm/dd/yy) | Pink Copy - Client

Relinquished by (NAME): Brett Thomas | Signature: [signature] | Date: 08 / 28 / 2020 (mm/dd/yy) | Yellow & White Copy - SGS

Revision # 1.5
Date of Issue: 13 Oct, 2019

Note: Submission of samples to SGS is acknowledgement that you have been provided direction on sample collection/handling and transportation of samples. (2) Submission of samples to SGS is considered authorization for completion of work. Signatures may appear on this form or be retained on file in the contract, or in an alternative format (e.g. shipping documents). (3) Results may be sent by email to an unlimited number of addresses for no additional cost. Fax is available upon request. This document is issued by the Company under its General Conditions of Service accessible at http://www.sgs.com/terms_and_conditions.htm. (Printed copies are available upon request.) Attention is drawn to the limitation of liability, indemnification and jurisdiction issues defined therein.

# Appendix B
# Record of Borehole Sheets and Laboratory Test Results for Previous Site Investigation

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

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 1

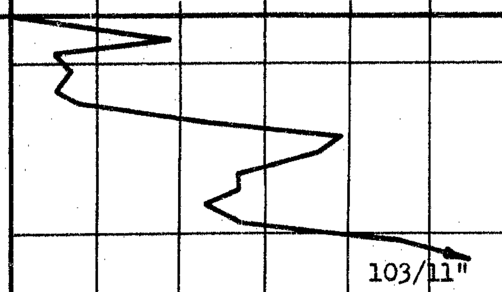
FOUNDATION SECTION

JOB 66-F-53
W.P. 634-64
DATUM Geodetic

LOCATION N 200,786.159, E210,719.093
BORING DATE May 31, 1966.
BOREHOLE TYPE Dynamic Cone Penetration

ORIGINATED BY D.W.
COMPILED BY D.W.
CHECKED BY K.G.S.

| SOIL PROFILE | | | SAMPLES | | | | DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 / SHEAR STRENGTH P.S.F. | LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_P$ WATER CONTENT — $w$ / WATER CONTENT % | BULK DENSITY $\gamma$ P.C.F. | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|
| ELEV. DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | BLOWS / FOOT | ELEV. SCALE | | | | |
| 1072.7 | Groundlevel | | | | | | | | | |
| 0.0 | | | | | | 1070 | | | | |
| | | | | | | 1060 | | | | |
| 1058.8 | | | | | | | | | | |
| 13.9 | End of borehole. | | | | | | | | | |
| | | | | | | 1050 | | | | |



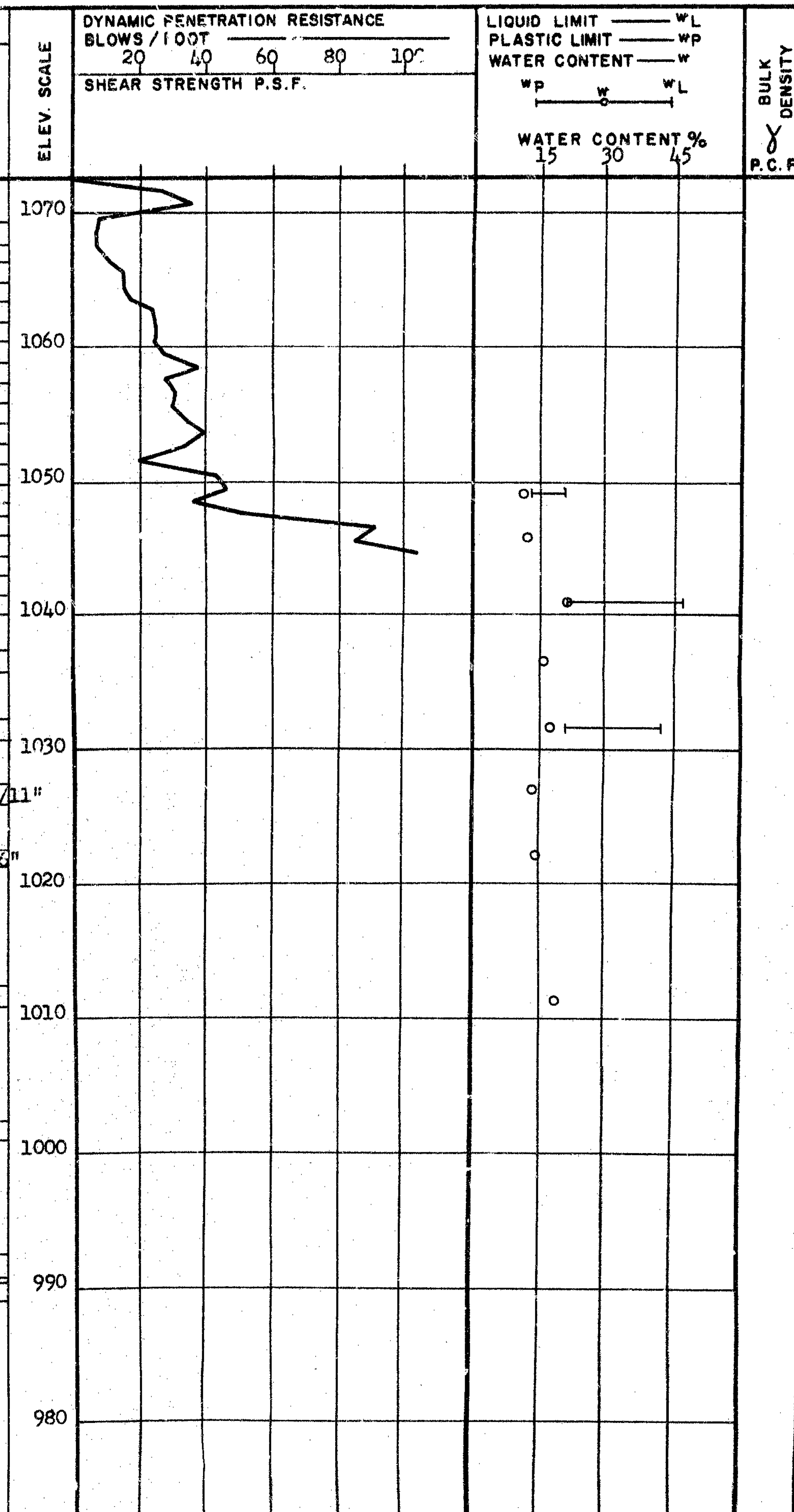
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 66-F-53 LOCATION N 200,887.529, E 210,709.963 ORIGINATED BY D.W.
W.P. 634-64 BORING DATE May 26, 1966. COMPILED BY D.W.
DATUM Geodetic BOREHOLE TYPE Penetration & Washboring. CHECKED BY K.G.S.

| SOIL PROFILE ELEV. DEPTH | DESCRIPTION | SAMPLES NUMBER | TYPE | BLOWS / FOOT | REMARKS |
|---|---|---|---|---|---|
| 1072.4 | Groundlevel | | | | |
| 0.0 | Sand (Topsoil) Loose | | | | |
| 1068.4 | | 1 | SS | 8 | Sa 80% Si 20% |
| 4.0 | Sand occasional trace of silt. Compact. | 2 | SS | 24 | |
| | | 3 | SS | 20 | G. W. L. El 1059.4 |
| | | 4 | SS | 25 | |
| | | 5 | SS | 10 | Sa96%Si 4% |
| | | 6 | SS | 30 | |
| 1052.4 | | | | | |
| 20.0 | Clayey silt with some sand and gravel Stiff to hard. | 7 | SS | 28 | |
| | | 8 | SS | 13 | Gr 14% Sa 27% |
| | | 9 | SS | 57 | Si 44% Cl 15% |
| 1045.4 | | | | | |
| 27.0 | Silty clay Hard Brownish grey. | 10 | SS | 71 | |
| | | 11 | SS | 44 | |
| | | 12 | SS | 194 | |
| | | 13 | SS | 88 | |
| 1028.4 | | | | | |
| 44.0 | Fine sandy silt to silty fine sand. Very dense. | 14 | SS | 100/11" | |
| | | 15 | SS | 85/6" | |
| 1013.4 | | | | | |
| 59.0 | Silty clay Hard Brownish grey. | 16 | SS | 120 | |
| | | 17 | SS | 92 | |
| | | 18 | SS | 46 | |
| 988.9 | | 19 | SS | 130 | |
| 83.5 | End of borehole. | | | | |



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

OFFICE REPORT ON SOIL EXPLORATION

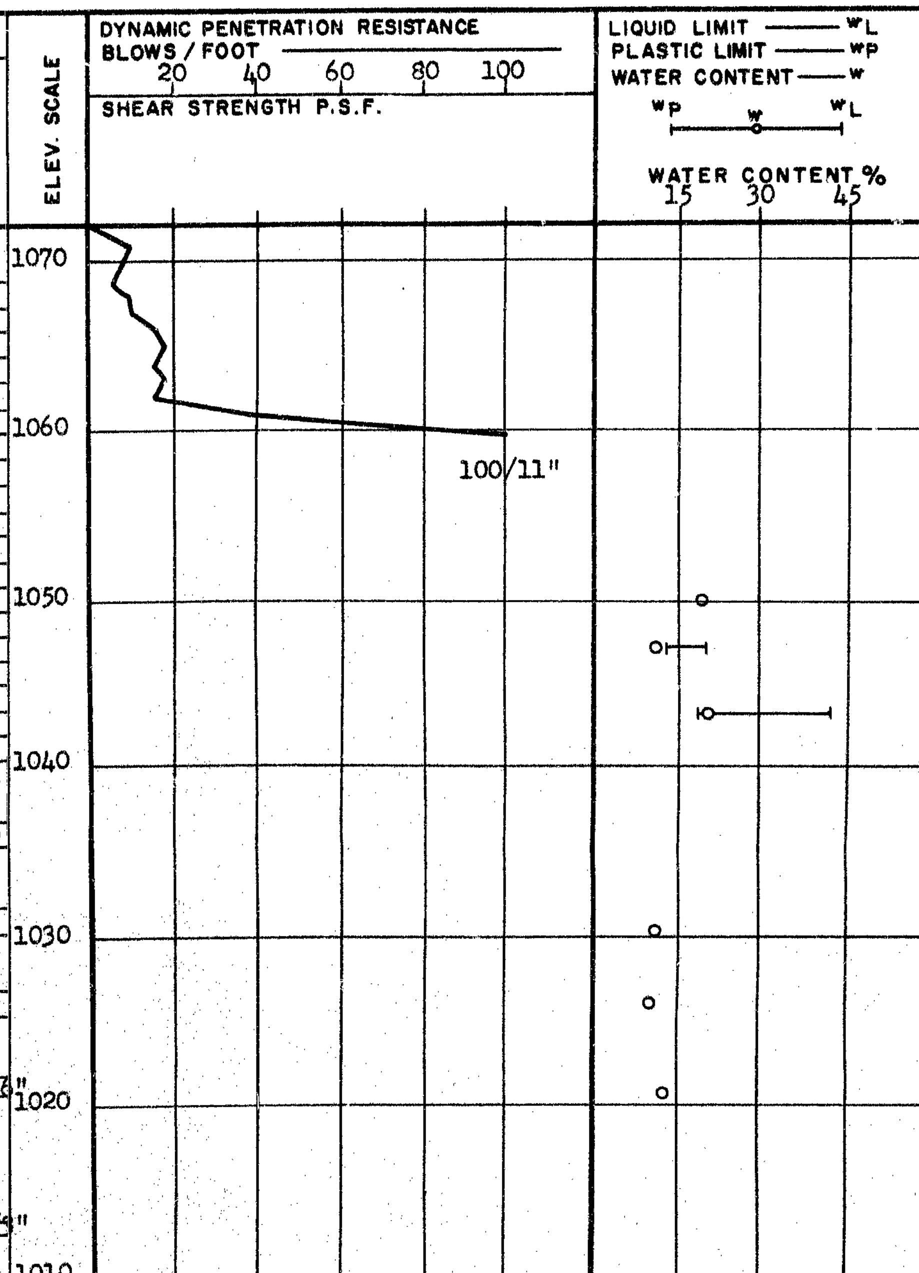
DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 66-F-53
W.P. 634-64
DATUM Geodetic
LOCATION N 200,803.248, E 210,761.731
BORING DATE May 31, 1966.
BOREHOLE TYPE Penetration & Washboring.
ORIGINATED BY D.W.
COMPILED BY D.W.
CHECKED BY K.G.S.

| ELEV. / DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | BLOWS / FOOT | REMARKS |
|---|---|---|---|---|---|
| 1071.9 | Groundlevel | | | | |
| 0.0 | Sand (Topsoil) | | | | |
| 1067.9 | Loose | 1 | SS | 2 | Sa 92% Si 8% |
| 4.0 | Sand Compact to dense. | 2 | SS | 10 | |
| | | 3 | SS | 36 | GWL Elev. 1059.1 |
| 1060.9 / 11.0 | Silty fine sand | | | | |
| 1057.9 | Compact | 4 | SS | 22 | |
| 14.0 | Gravelly sand Dense to very dense | 5 | SS | 45 | Gr 47% Sa 46% Si 7% |
| | | 6 | SS | 62 | |
| 1051.9 / 20.0 | Clayey silt Hard | 7 | SS | 27 | |
| 1045.9 | Grey | 8 | SS | 31 | Gr 7% Sa 19% Si 54% Cl 20% |
| 26.0 | Silty clay Hard Brownish grey | 9 | SS | 49 | |
| | | 10 | SS | 108 | |
| | | 11 | SS | 60 | |
| 1032.9 / 39.0 | Silty fine sand to fine sandy silt. Very dense. | 12 | SS | 99 | Sa 22% Si 78% |
| | | 13 | SS | 158 | |
| | | 14 | SS | 87/6" | |
| 1012.4 | | 15 | SS | 50/3" | |
| 59.5 | End of borehole. | | | | |



FORM DB-MT-126 (FORMERLY DB-ML-126)
66-1831

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 4

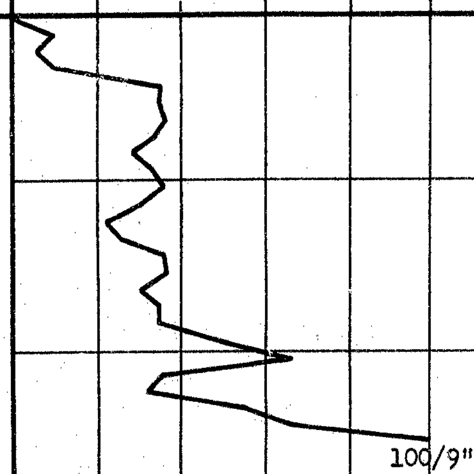
FOUNDATION SECTION

JOB 66-F-53
W.P. 634-64
DATUM Geodetic

LOCATION N 200,890.443, E 210,747.672
BORING DATE May 30, 1966.
BOREHOLE TYPE Dynamic Cone Penetration.

ORIGINATED BY D.W.
COMPILED BY D.W.
CHECKED BY K.G.S.

| SOIL PROFILE: ELEV. / DEPTH | SOIL PROFILE: DESCRIPTION | STRAT. PLOT | SAMPLES: NUMBER | SAMPLES: TYPE | SAMPLES: BLOWS / FOOT | ELEV. SCALE | DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT (20, 40, 60, 80, 100); SHEAR STRENGTH P.S.F. | LIQUID LIMIT — $w_L$; PLASTIC LIMIT — $w_P$; WATER CONTENT — $w$; WATER CONTENT % | BULK DENSITY $\gamma$ P.C.F. | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|
| 1069.3 / 0.0 | Groundlevel | | | | | | | | | |
| | | | | | | 1060 | | | | |
| | | | | | | 1050 | | | | |
| 1044.5 / 24.8 | End of borehole. | | | | | | 100/9" | | | |
| | | | | | | 1040 | | | | |



FORM DB-MT-126 (FORMERLY DB-ML-126)
66-1831

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 5

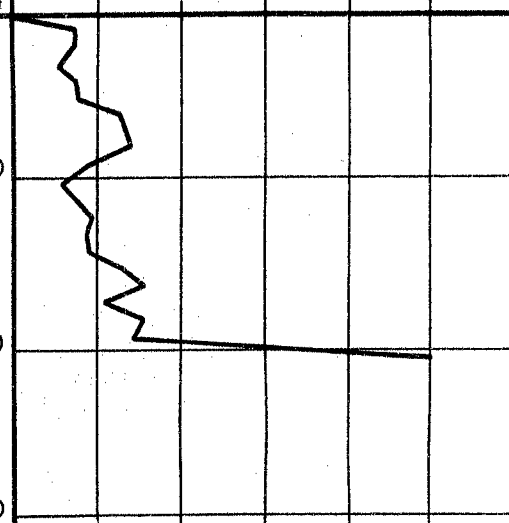
FOUNDATION SECTION

JOB 66-F-53 | LOCATION N 200,821,407, E210,811,176 | ORIGINATED BY D.W.

W.P. 634-64 | BORING DATE May 31, 1966. | COMPILED BY D.W.

DATUM Geodetic | BOREHOLE TYPE Dynamic Cone Penetration | CHECKED BY K.G.S.

| SOIL PROFILE | | | SAMPLES | | | | DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 SHEAR STRENGTH P.S.F. | LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_P$ WATER CONTENT $w$ WATER CONTENT % | BULK DENSITY $\gamma$ P.C.F. | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|
| ELEV. / DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | BLOWS / FOOT | ELEV. SCALE | | | | |
| 1069.8 | Groundlevel | | | | | | | | | |
| 0.0 | | | | | | 1060 | | | | |
| 1049.8 | | | | | | 1050 | | | | |
| 20.0 | End of borehole. | | | | | 1040 | | | | |



FORM OB-MT-126 (FORMERLY OB-ML-126)
66-1931

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 6

FOUNDATION SECTION

JOB 66-F-53
W.P. 634-64
DATUM Geodetic
LOCATION N 200,919.964, E 210,802,832
BORING DATE May 30, 1966.
BOREHOLE TYPE Penetration & Washboring.
ORIGINATED BY D.W.
COMPILED BY D.W.
CHECKED BY K.G.S.

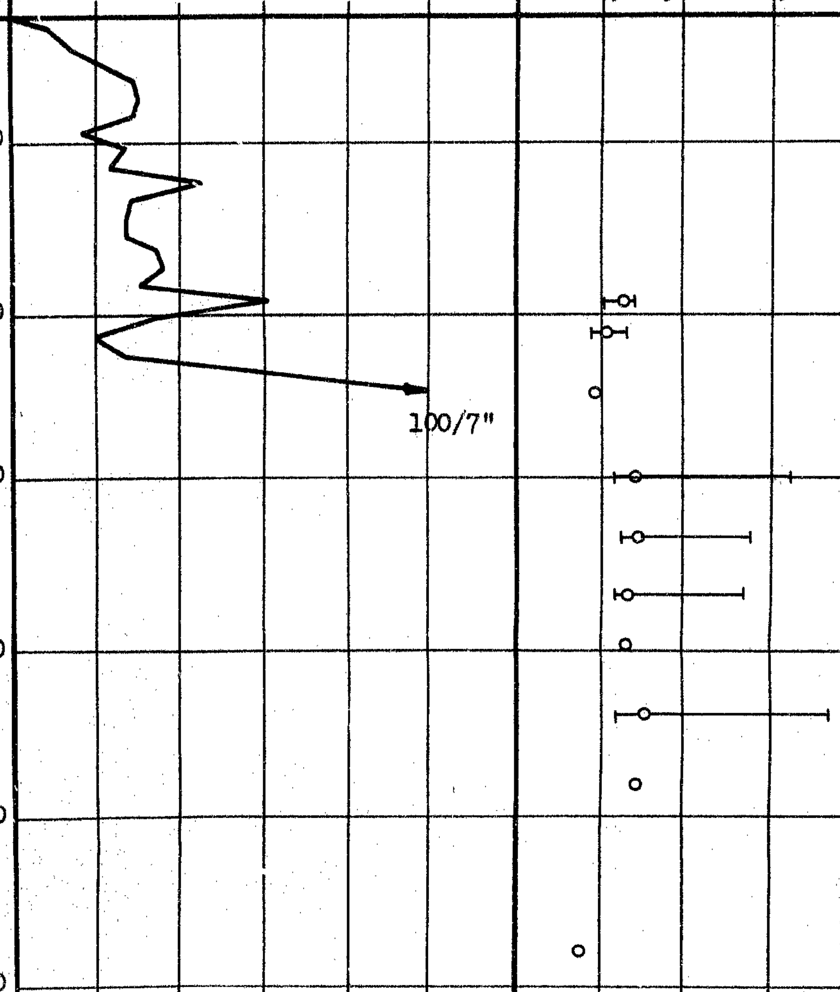
| ELEV. / DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | BLOWS / FOOT | ELEV. SCALE | REMARKS |
|---|---|---|---|---|---|---|
| 1067.7 | Groundlevel | | | | | |
| 0.0 / 1064.7 | Sand (Topsoil) Loose | | | | | |
| 3.0 | Sand with trace of silt. Compact. | 1 | SS | 24 | 1060 | GWL El. 1059.2 |
| | | 2 | SS | 28 | | Gr 2% Sa 90% Si 8% |
| 1053.7 / 14.0 | Clayey silt. Very stiff Grey | 3 | SS | 26 | 1050 | |
| | | 4 | SS | 28 | | |
| 1047.7 / 20.0 | Fine sandy silt. Very dense | 5 | SS | 157/11" | | Sa 42% Si 58% |
| | | 6 | SS | 67 | | |
| 1042.5 / 25.2 | Silty clay with trace of sand and gravel. Hard. | 7 | SS | 34 | 1040 | |
| | | 8 | SS | 38 | | |
| | | 9 | SS | 60 | | |
| 1032.7 / 35.0 | Silty clay Hard Brownish grey | 10 | SS | 85 | 1030 | |
| | | 11 | SS | 43 | | |
| | | 12 | SS | 60 | 1020 | |
| 1015.7 / 52.0 | Silty fine sand Very dense | 13 | SS | 131/9" | 1010 | |
| 1011.4 / 56.3 | End of borehole. | | | | | |

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F.

LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_P$ WATER CONTENT $w$

WATER CONTENT % 15 30 45

BULK DENSITY $\gamma$ P.C.F.



FORM OB-MT-126 (FORMERLY OB-ML-126)
66-1631

OFFICE REPORT ON SOIL EXPLORATION

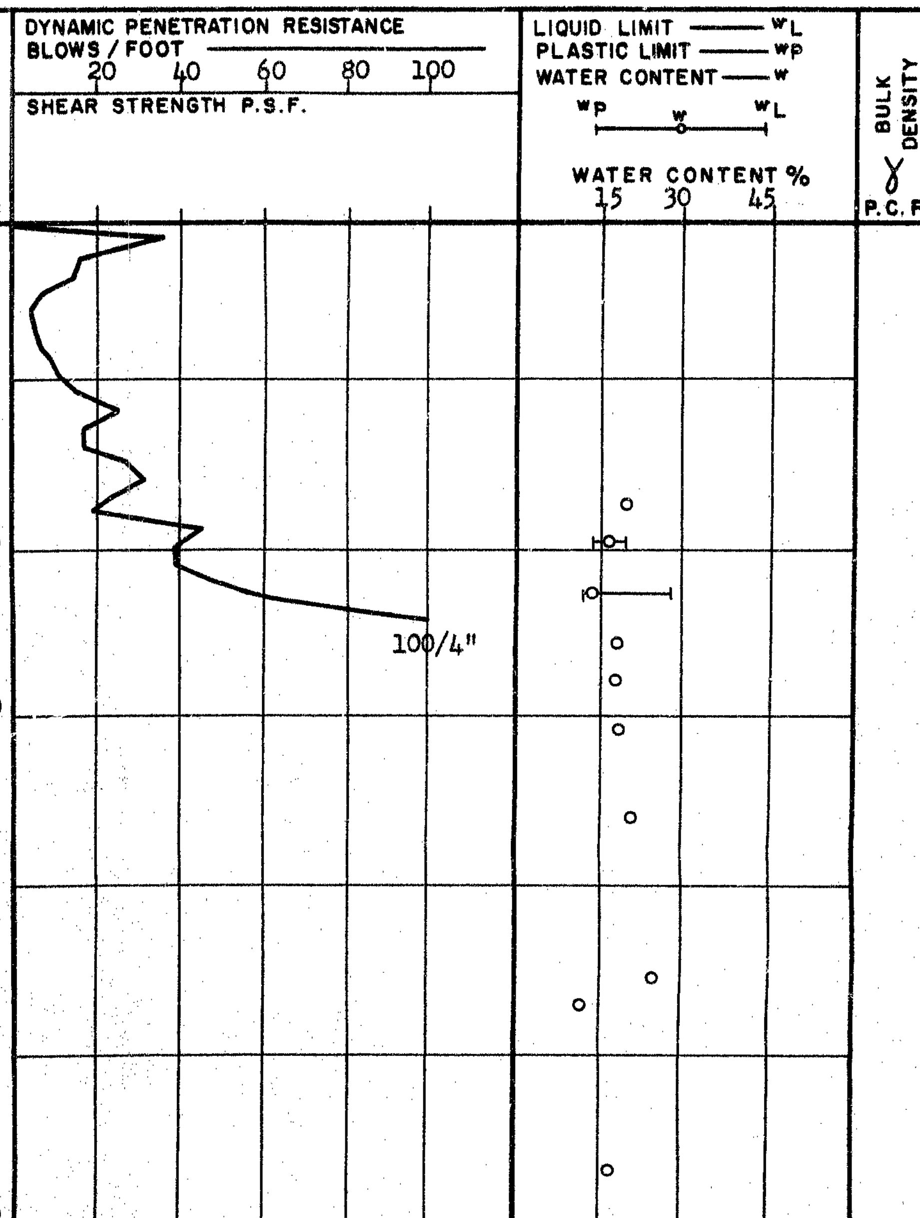
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 7

FOUNDATION SECTION

JOB 66-F-53 LOCATION N 200,844.706, E 210,861.156 ORIGINATED BY D.W.
W.P. 634-64 BORING DATE May 31, 1966. COMPILED BY D.W.
DATUM Geodetic BOREHOLE TYPE Penetration & Washboring. CHECKED BY K.G.S.

| ELEV. DEPTH | SOIL PROFILE DESCRIPTION | SAMPLES NUMBER | TYPE | BLOWS / FOOT | ELEV. SCALE | REMARKS |
|---|---|---|---|---|---|---|
| 1069.1 | Groundlevel | | | | | |
| 0.0 | Sand (Topsoil) Loose | 1 | SS | 6 | | Sa 88% Si 12% |
| 1062.6 / 6.5 | Sand with trace of silt. Compact. | 2 | SS | 20 | 1060 | GWL El. 1058.6 |
| 1055.1 / 14.0 | Clayey silt with trace of sand and gravel. Very stiff to hard. Grey | 3 | SS | 19 | | |
| | | 4 | SS | 16 | 1050 | |
| | | 5 | SS | 34 | | |
| 1046.1 / 23.0 | Silty fine sand. Very dense. | 6 | SS | 100/11" | | Sa 67% Si 32% |
| | | 7 | SS | 100/9" | 1040 | |
| | | 8 | SS | 52/3" | | |
| 1036.1 / 33.0 | Silty clay. Hard. Brownish grey. | 9 | SS | 80 | | |
| | | 10 | SS | 61 | 1030 | |
| | | 11 | SS | 97 | | |
| 1023.1 / 46.0 | Fine sandy silt. Very dense. | 12 | SS | 110/5" | 1020 | Sa 28% Si 72% |
| 1013.2 / 55.9 | End of borehole. | | | | 1010 | |



FORM OS-MT-126 (FORMERLY OS-ML-126)
66-1831

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 8

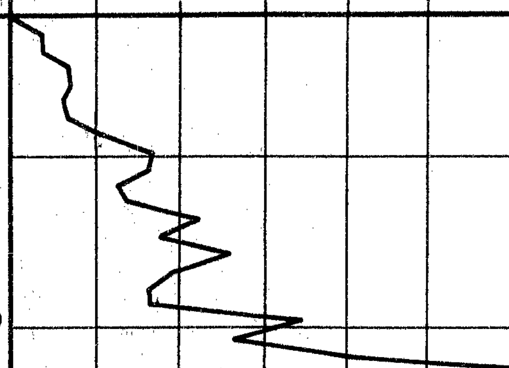
FOUNDATION SECTION

JOB 66-F-53
W.P. 634-64
DATUM Geodetic

LOCATION N 200,939.749, E 210,847.402
BORING DATE June 6, 1966.
BOREHOLE TYPE Dynamic Cone Penetration.

ORIGINATED BY D.W.
COMPILED BY D.W.
CHECKED BY K.G.S.

| SOIL PROFILE: ELEV. / DEPTH | SOIL PROFILE: DESCRIPTION | STRAT. PLOT | SAMPLES: NUMBER | SAMPLES: TYPE | SAMPLES: BLOWS/FOOT | ELEV. SCALE | DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 — SHEAR STRENGTH P.S.F. | LIQUID LIMIT — $w_L$, PLASTIC LIMIT — $w_P$, WATER CONTENT — $w$; WATER CONTENT % | BULK DENSITY γ P.C.F. | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|
| 1068.1 | Groundlevel | | | | | | | | | |
| 0.0 | | | | | | | | | | |
| | | | | | | 1060 | | | | |
| | | | | | | 1050 | | | | |
| 1047.3<br>20.8 | End of borehole. | | | | | | 120/9" | | | |



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

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 9

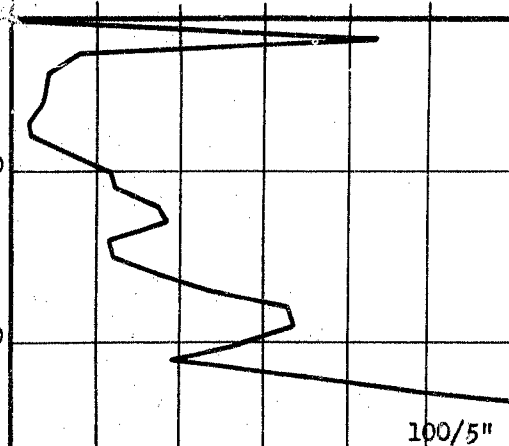
FOUNDATION SECTION

JOB 66-F-53
W.P. 634-64
DATUM Geodetic

LOCATION N 200,873.400, E 210,912.942
BORING DATE June 2, 1966
BOREHOLE TYPE Dynamic Cone Penetration.

ORIGINATED BY D.W.
COMPILED BY D.W.
CHECKED BY K.G.S.

| SOIL PROFILE: ELEV. / DEPTH | SOIL PROFILE: DESCRIPTION | STRAT. PLOT | SAMPLES: NUMBER | SAMPLES: TYPE | SAMPLES: BLOWS / FOOT | ELEV. SCALE | DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT (20, 40, 60, 80, 100); SHEAR STRENGTH P.S.F. | LIQUID LIMIT $w_L$, PLASTIC LIMIT $w_P$, WATER CONTENT $w$; WATER CONTENT % | BULK DENSITY γ P.C.F. | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|
| 1069.0 | Groundlevel | | | | | | | | | |
| | | | | | | 1060 | | | | |
| | | | | | | 1050 | | | | |
| 1046.5 / 22.5 | End of borehole. | | | | | | 100/5" | | | |
| | | | | | | 1040 | | | | |



FORM DB-MT-126 (FORMERLY DB-ML-126)
65-1631

# OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

## RECORD OF BOREHOLE NO. 10

FOUNDATION SECTION

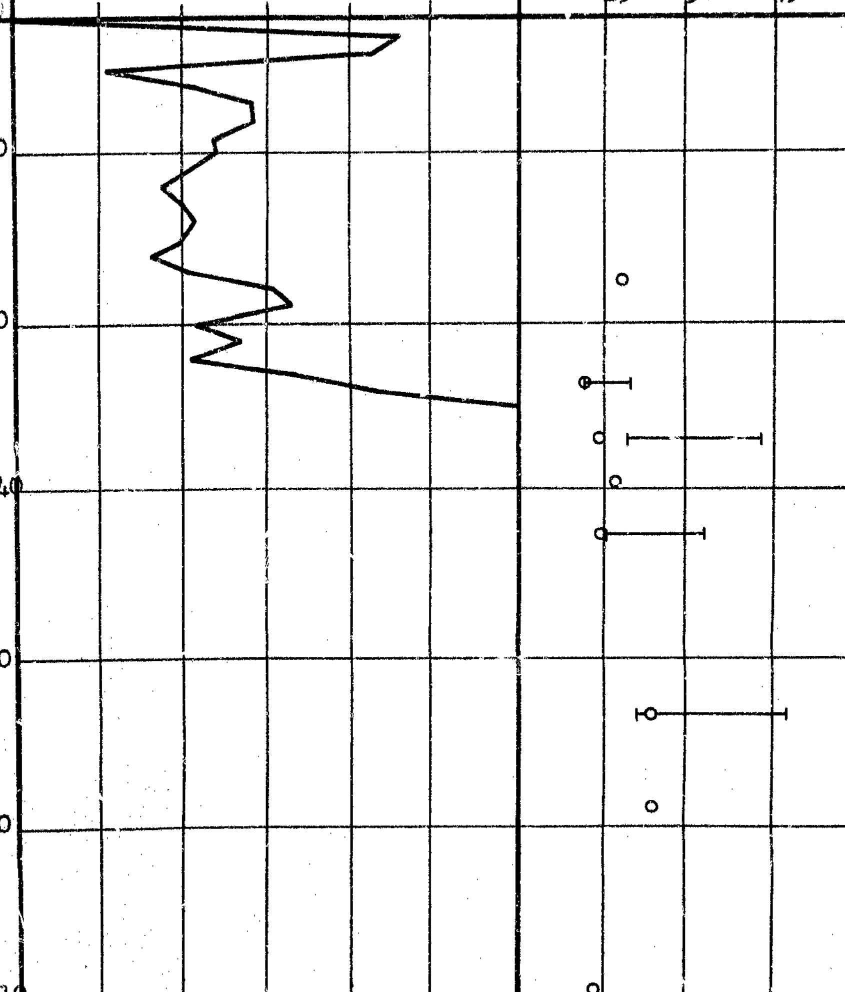
JOB 66-F-53 | LOCATION N 200,955.200, E 210,895.815 | ORIGINATED BY D.W.
W.P. 634-64 | BORING DATE June 3, 1966. | COMPILED BY D.W.
DATUM Geodetic | BOREHOLE TYPE Penetration & Washboring | CHECKED BY K.G.S.

| ELEV. / DEPTH | SOIL PROFILE – DESCRIPTION | SAMPLE NUMBER | TYPE | BLOWS / FOOT | ELEV. SCALE | REMARKS |
|---|---|---|---|---|---|---|
| 1067.9 | Groundlevel | | | | | |
| 0.0 / 1064.9 | Fill gravelly sand. | | | | | Sa 90% Si 10% |
| 3.0 | Sand with trace of silt. Loose to compact. | 1 | SS | 9 | 1060 | GWL El. 1056.3 |
| | | 2 | SS | 29 | | |
| 1053.9 / 14.0 | Fine sandy silt. Dense | 3 | SS | 34 | 1050 | |
| | | 4 | SS | 29 | | |
| 1047.9 / 20.0 | Clayey silt with some sand & gravel. Hard. | 5 | SS | 64 | | |
| 1044.9 / 23.0 | Clayey silt to silty clay. Hard. Brownish grey. | 6 | SS | 99 | | |
| | | 7 | SS | 77/9" | 1040 | |
| | | 8 | SS | 64 | | |
| | | 9 | SS | 73 | 1030 | |
| | | 10 | SS | 41 | | |
| | | 11 | SS | 45 | 1020 | |
| 1011.4 | Fine sandy silt. Very dense. | 12 | SS | 70 | | |
| 1010.6 / 57.5 | End of borehole. | 13 | SS | 123/6" | 1010 | Sa 5% Si 95% |

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20 40 60 80 100
SHEAR STRENGTH P.S.F.

LIQUID LIMIT — $w_L$
PLASTIC LIMIT — $w_P$
WATER CONTENT — $w$
WATER CONTENT %: 15 30 45

BULK DENSITY γ P.C.F.



FORM OB-MT-126 (FORMERLY OB-ML-126)
66-1631

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

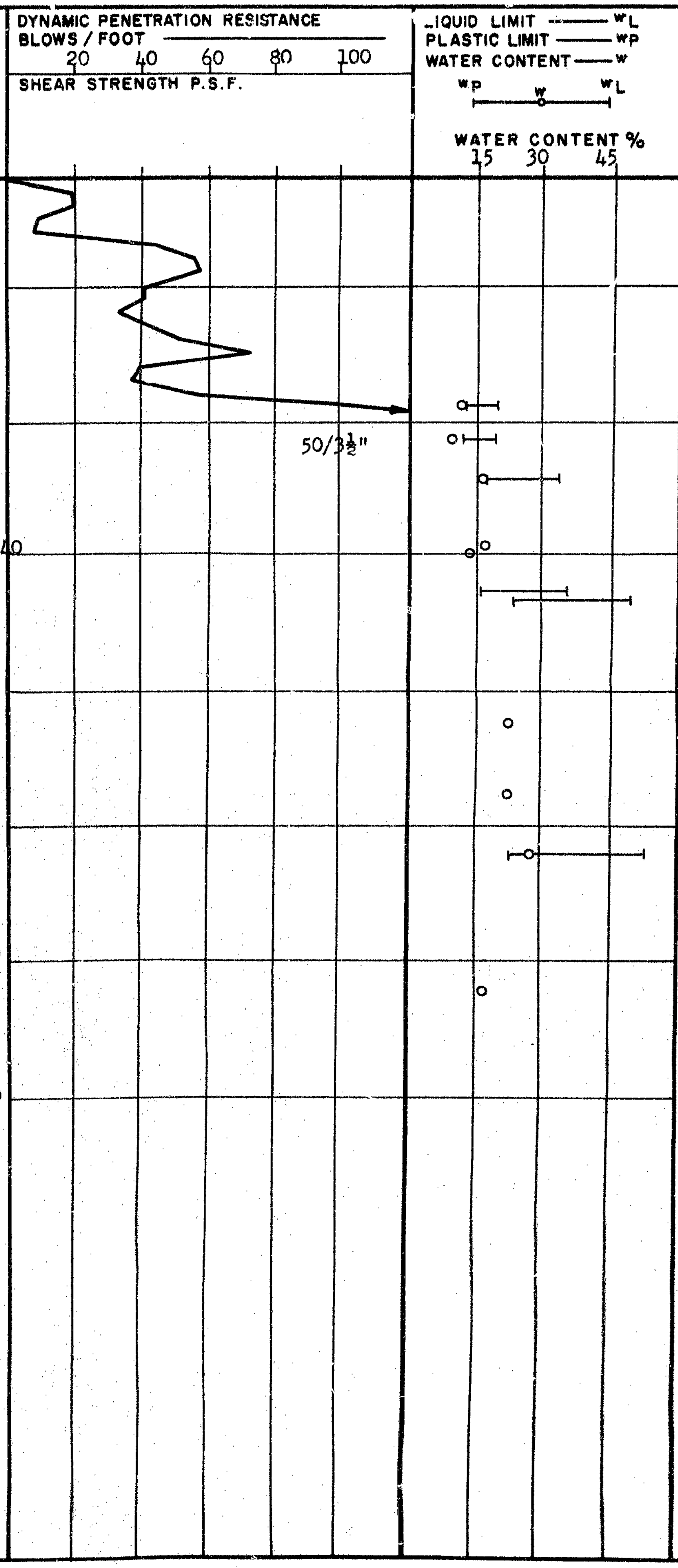
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 11

FOUNDATION SECTION

| | | | |
|---|---|---|---|
| JOB 66-F-53 | LOCATION N 200,883.088, E 210,947.710 | ORIGINATED BY D.W. |
| W.P. 634-64 | BORING DATE May 31, 1966. | COMPILED BY D.W. |
| DATUM Geodetic | BOREHOLE TYPE Penetration & Washboring | CHECKED BY K.G.S. |

| ELEV. / DEPTH | SOIL PROFILE DESCRIPTION | SAMPLE NUMBER | TYPE | BLOWS / FOOT | REMARKS |
|---|---|---|---|---|---|
| 1068.2 | Groundlevel | | | | |
| 0.0 | Sand (Topsoil) Loose | 1 | SS | 6 | |
| 1063.2 / 5.0 | Sand with trace of silt. Compact. | 2 | SS | 13 | Sa 83% Si 17% |
| | | 3 | SS | 23 | GWL El. 1055.7 |
| 1057.2 / 11.0 | Silty fine sand. Dense | 4 | SS | 47 | |
| 1054.2 / 14.0 | Clayey silt with some sand and gravel. Hard. | 5 | SS | 32 | |
| | | 6 | SS | 41 | |
| | | 7 | SS | 57 | |
| 1045.2 / 23.0 | Fine sandy silt Very dense | 8 | SS | 70 | Sa 25% Si 75% |
| | | 9 | SS | 100/3½" | |
| 1039.2 / 29.0 | Silty clay. Hard. Brownish grey. | 10 | SS | 132 | Sa 1% Si 54% Cl 45% |
| | | 11 | SS | 131 | |
| | | 12 | SS | 62 | |
| | | 13 | SS | 64 | |
| | | 14 | SS | 59 | |
| 1009.2 | | | | | |
| 1007.2 / 61.0 | Fine sandy silt. Very dense. | 15 | SS | 62/6" | Sa 26% Si 74% |
| | End of borehole. | | | | |




ELEV. SCALE: 1060, 1050, 1040, 1030, 1020, 1010, 1000

BULK DENSITY γ P.C.F.

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

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 12

FOUNDATION SECTION

JOB 66-F-53 LOCATION N 200,967.693, E 210,949.158 ORIGINATED BY D.W.

W.P. 634-64 BORING DATE June 6, 1966 COMPILED BY D.W.

DATUM Geodetic BOREHOLE TYPE Dynamic Cone Penetration. CHECKED BY K.G.S.

| SOIL PROFILE | | | SAMPLES | | | ELEV. SCALE | DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 / SHEAR STRENGTH P.S.F. | LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_P$ WATER CONTENT — $w$ / WATER CONTENT % | BULK DENSITY $\gamma$ P.C.F. | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|
| ELEV. DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | BLOWS / FOOT | | | | | |
| 1067.6 | Groundlevel | | | | | | | | | |
| 0.0 | | | | | | | | | | |
| | | | | | | 1060 | | | | |
| | | | | | | 1050 | | | | |
| 1041.6 | | | | | | | | | | |
| 26.0 | End of borehole. | | | | | 1040 | | | | |

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

OFFICE REPORT ON SOIL EXPLORATION

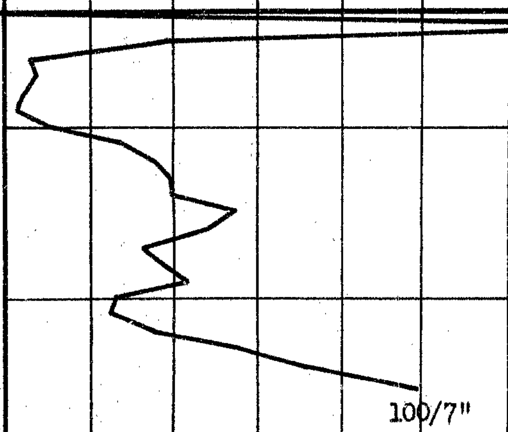
DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 13

FOUNDATION SECTION

JOB 66-F-53
W.P. 634-64
DATUM Geodetic
LOCATION N 200,914.762, E 211,000.220
BORING DATE June 2,1966
BOREHOLE TYPE Dynamic Cone Penetration
ORIGINATED BY D.W.
COMPILED BY D.W.
CHECKED BY K.G.S.

| SOIL PROFILE | | | SAMPLES | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| ELEV. / DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | BLOWS / FOOT | ELEV. SCALE | DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 / SHEAR STRENGTH P.S.F. | LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_P$ WATER CONTENT — $w$ / WATER CONTENT % | BULK DENSITY γ P.C.F. | REMARKS |
| 1067.0 | Groundlevel | | | | | | | | | |
| 0.0 | | | | | | 1060 | | | | |
| | | | | | | 1050 | | | | |
| 1044.4 / 22.6 | End of borehole. | | | | | 1040 | | | | |



FORM DB-MT-126 (FORMERLY DB-ML-126)
66-1831

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

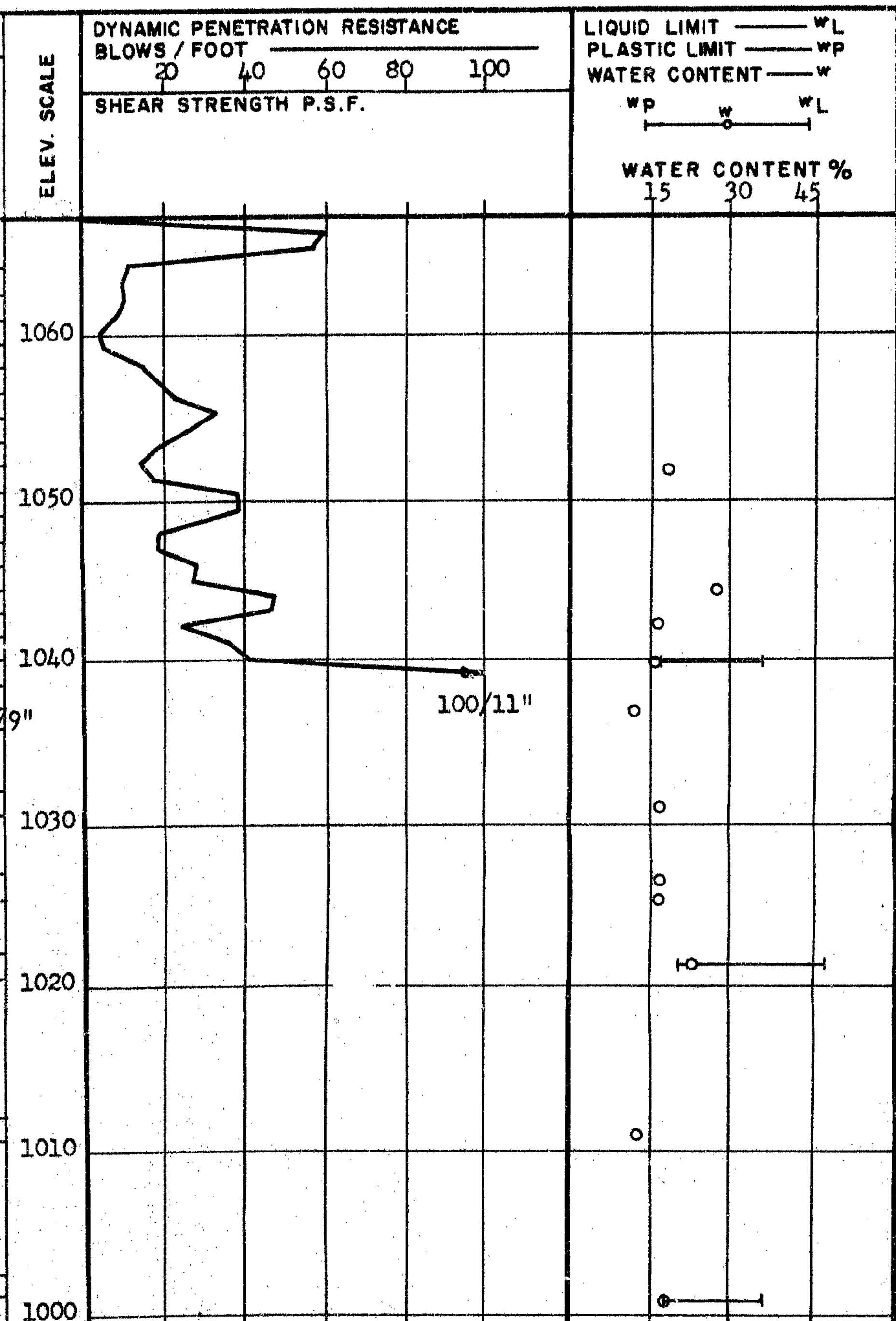
# RECORD OF BOREHOLE NO. 14

FOUNDATION SECTION

| | | | |
|---|---|---|---|
| JOB 66-F-53 | LOCATION N 200,997.938, E 210,999.324 | ORIGINATED BY D.W. |
| W.P. 634-64 | BORING DATE June 2, 1966 | COMPILED BY D.W. |
| DATUM Geodetic | BOREHOLE TYPE Penetration & Washboring. | CHECKED BY K.G.S. |

## SOIL PROFILE / SAMPLES

| ELEV. DEPTH | DESCRIPTION | NUMBER | TYPE | BLOWS / FOOT |
|---|---|---|---|---|
| 1067.2 | Groundlevel | | | |
| 0.0 | Sand (Topsoil) Compact | 1 | SS | 11 |
| 1062.2 / 5.0 | Sand with trace of silt. Compact. | 2 | SS | 14 |
| | | 3 | SS | 18 |
| | | 4 | SS | 15 |
| | | 5 | SS | 15 |
| | | 6 | SS | 17 |
| | | 7 | SS | 20 |
| | | 8 | SS | 28 |
| 1041.2 / 26.0 | Silty clay with trace of sand. Hard. | 9 | SS | 130 |
| 1038.2 / 29.0 | Sand with some silt. Very dense. | 10 | SS | 100/9" |
| | | 11 | SS | 62 |
| 1026.2 / 41.0 | Silty clay. Hard. | 12 | SS | 93 |
| | | 13 | SS | 47 |
| 1015.2 / 52.0 | Fine sandy silt. Very dense. | 14 | SS | 109 |
| 1006.2 / 61.0 | Silty clay Hard Brownish grey | 15 | SS | 120 |
| 1000.7 / 66.5 | End of borehole. | | | |




## REMARKS

Sa 89%
Si 11%

GWL El. 1054.2

Sa 93%
Si 7%

Gr 2%
Sa 97%
Si 1%

FORM DB-MT-126 (FORMERLY DB-ML-126)
66-1631

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 16

FOUNDATION SECTION

JOB 66-F-53
W.P. 634-64
DATUM Geodetic
LOCATION N 201,116.543, E 210,741.917
BORING DATE June 3, 1966
BOREHOLE TYPE Penetration & Washboring
ORIGINATED BY D.W.
COMPILED BY D.W.
CHECKED BY K.G.S.

| ELEV. / DEPTH | DESCRIPTION (SOIL PROFILE) | REMARKS |
|---|---|---|
| 1065.0 | Groundlevel | |
| 0.0 – 1056.0 | Sand<br>Compact | |
| 9.0 – 1036.0 | Clayey silt to silty clay with some sand and gravel.<br>Very stiff to hard<br>Brownish grey. | Gr 7%<br>Sa 28%<br>Si 46%<br>Cl 19%<br><br>Sa 2%<br>Si 41%<br>Cl 57% |
| 29.0 – 1024.0 | Silty clay<br>Hard<br>Brownish grey. | |
| 41.0 – 1008.5 | Silty fine sand.<br>Very dense | |
| 56.5 | End of borehole. | |

| SAMPLES NUMBER | TYPE | BLOWS / FOOT |
|---|---|---|
| 1 | SS | 17 |
| 2 | SS | 26 |
| 3 | SS | 62 |
| 4 | SS | 111 |
| 5 | SS | 126 |
| 6 | SS | 87 |
| 7 | SS | 85 |
| 8 | SS | 83 |
| 9 | SS | 39 |
| 10 | SS | 105 |
| 11 | SS | 93/6" |
| 12 | SS | 116 |

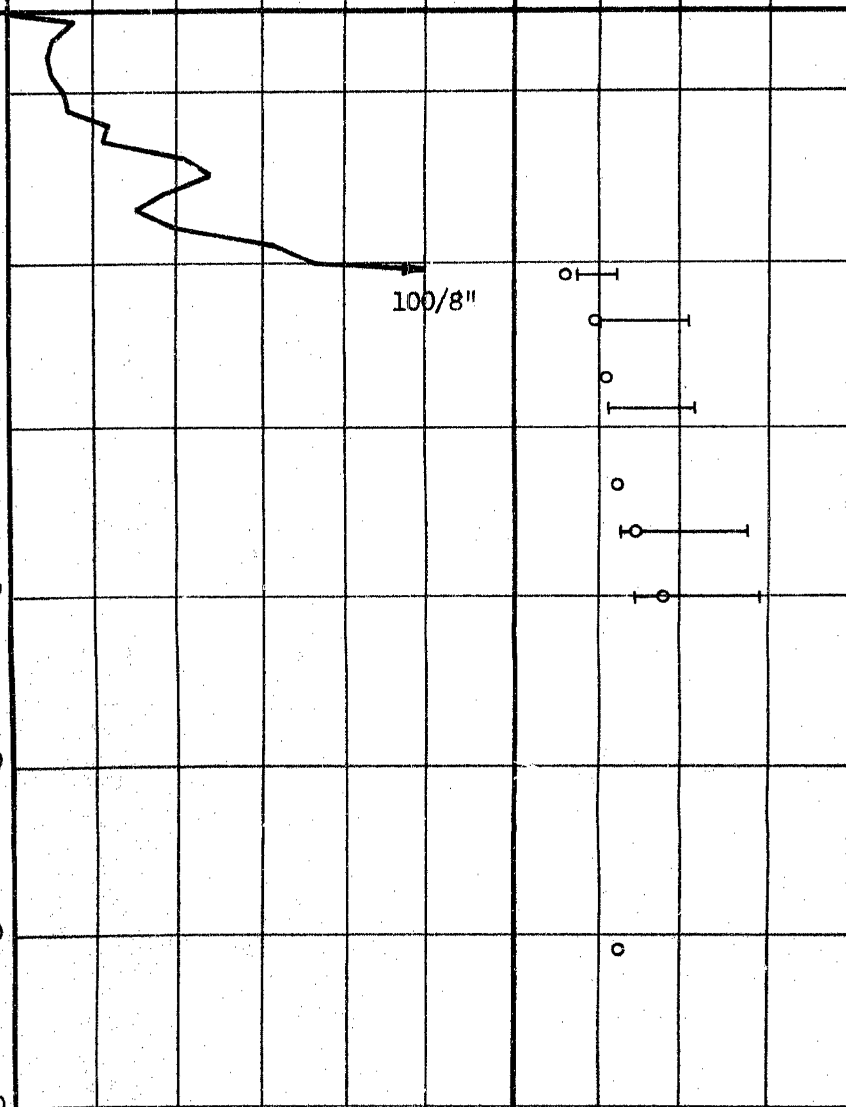
ELEV. SCALE: 1060, 1050, 1040, 1030, 1020, 1010, 1000

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F.

LIQUID LIMIT — $w_L$
PLASTIC LIMIT — $w_P$
WATER CONTENT — $w$

WATER CONTENT % 15 30 45

BULK DENSITY γ P.C.F.



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

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

# RECORD OF BOREHOLE NO. 17

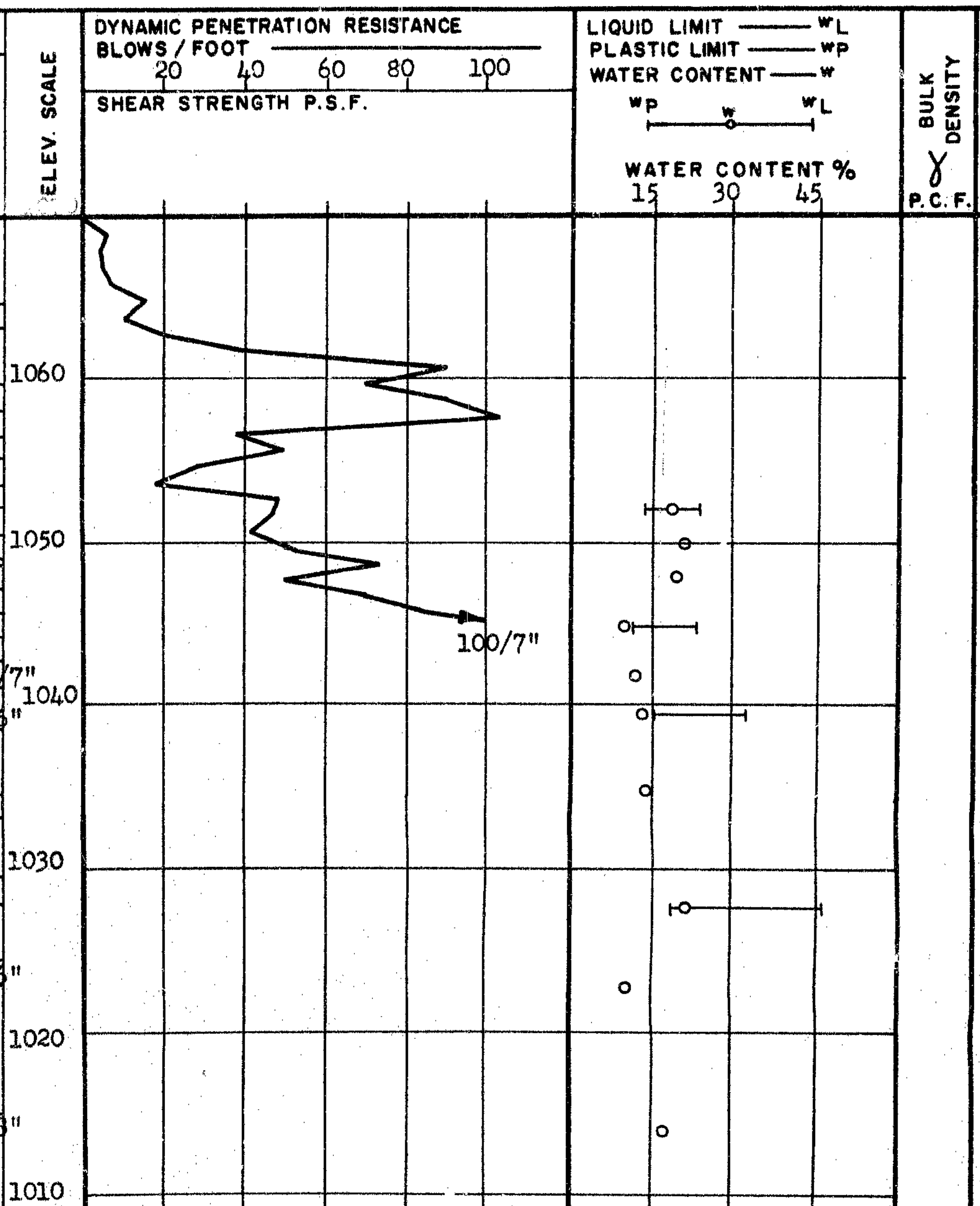
FOUNDATION SECTION

JOB 66-F-53
W.P. 634-64
DATUM Geodetic

LOCATION N 200,754.293, E 210,935.966
BORING DATE June 2, 1966
BOREHOLE TYPE Penetration & Washboring

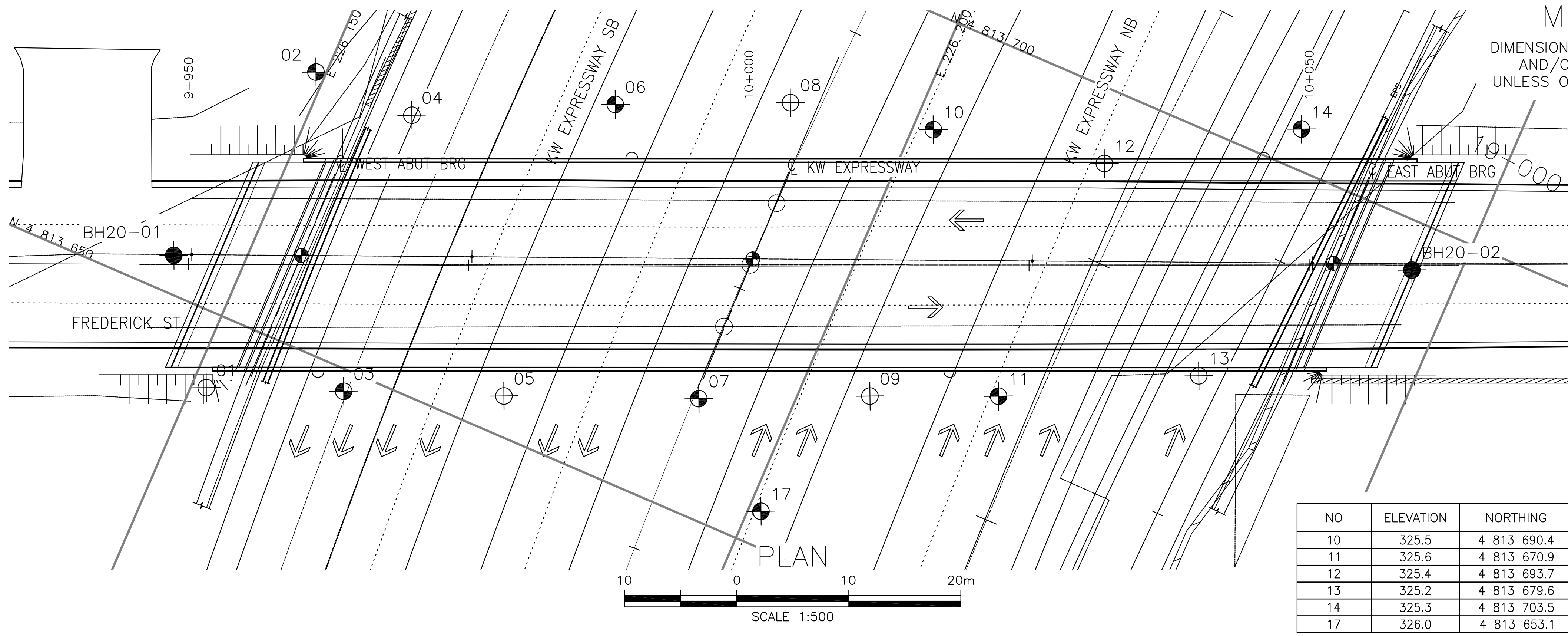
ORIGINATED BY D.W.
COMPILED BY D.W.
CHECKED BY K.G.S.

| ELEV. DEPTH | SOIL PROFILE DESCRIPTION | SAMPLE NUMBER | TYPE | BLOWS / FOOT | REMARKS |
|---|---|---|---|---|---|
| 1069.6 | Groundlevel | | | | |
| 0.0 | Sand<br>Loose to v. dense | 1 | SS | 9 | Sa 91%<br>Si 9% |
| | | 2 | SS | 70 | GWL El. 1057.8 |
| | | 3 | SS | 28 | |
| 1054.6<br>15.0 | Clayey silt<br>Very stiff to hard. | 4 | SS | 34 | Sa 1%<br>Si 80%<br>Cl 19% |
| | | 5 | SS | 22 | |
| 1049.1<br>20.5 | Fine sandy silt, v. dense | 6 | SS | 27 | |
| 1046.6<br>23.0 | Clayey silt to silty clay.<br>Hard.<br>Brownish grey. | 7 | SS | 53 | |
| | | 8 | SS | 150/7" | |
| | | 9 | SS | 60/6" | |
| | | 10 | SS | 88 | |
| | | 11 | SS | 76 | |
| 1025.[illegible]<br>44.[illegible] | [illegible]ne sandy silt.<br>very dense | 12 | SS | 68/6" | Sa 28%<br>Si 72% |
| | | 13 | SS | 50/3" | |
| 1013.8<br>55.8 | End of borehole. | | | | |

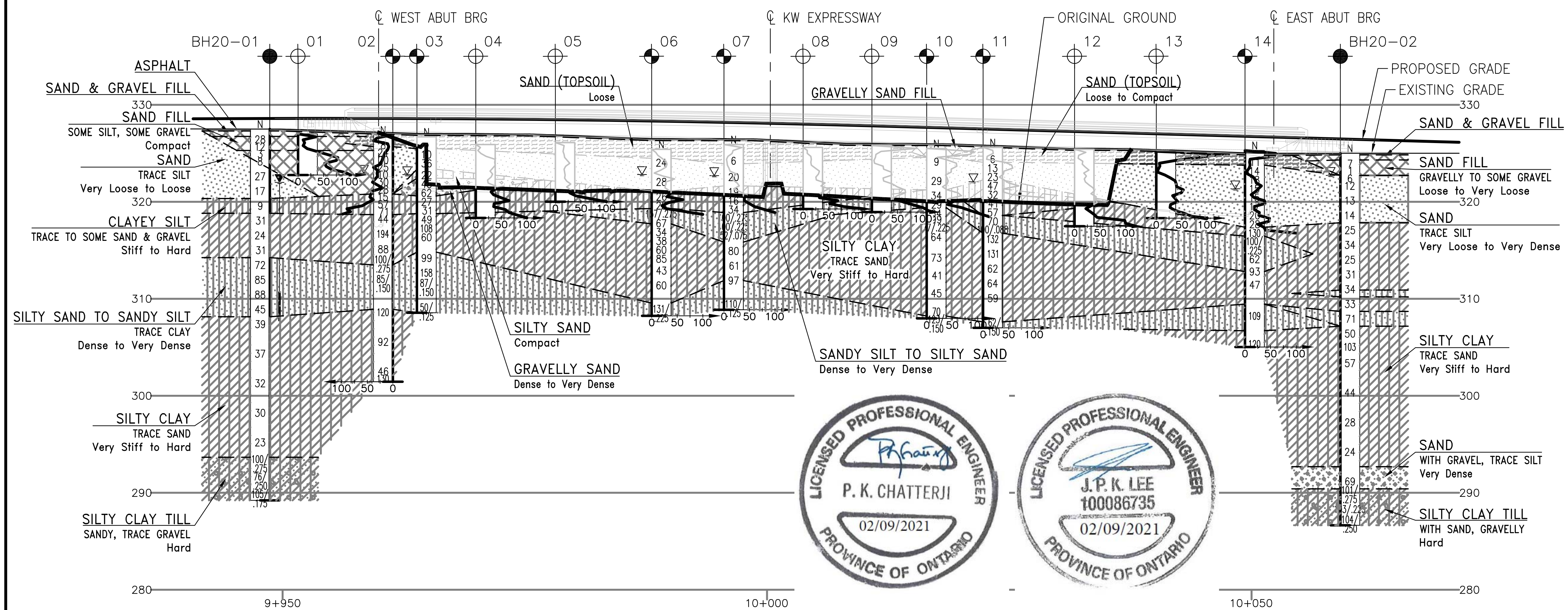


## Appendix C
## Borehole Locations and Soil Strata Drawings

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN




| NO | ELEVATION | NORTHING | EASTING |
|---|---|---|---|
| 10 | 325.5 | 4 813 690.4 | 226 201.8 |
| 11 | 325.6 | 4 813 670.9 | 226 216.5 |
| 12 | 325.4 | 4 813 693.7 | 226 217.0 |
| 13 | 325.2 | 4 813 679.6 | 226 232.2 |
| 14 | 325.3 | 4 813 703.5 | 226 231.9 |
| 17 | 326.0 | 4 813 653.1 | 226 201.1 |




LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
02/09/2021
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
J. P. K. LEE
100086735
02/09/2021
PROVINCE OF ONTARIO

PROFILE ALONG ℄ FREDERICK STREET

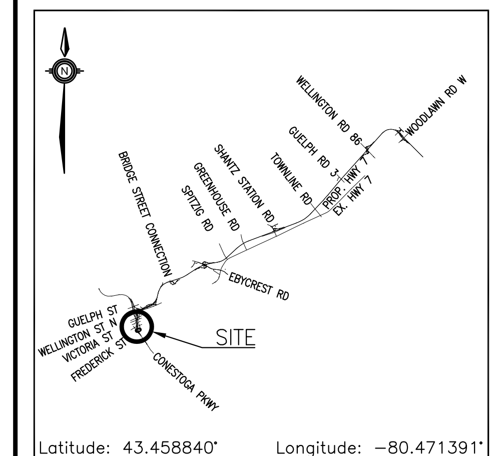
H 1:500
V 1:500

CONT No
GWP No 408-88-00

SHEET

HIGHWAY 7
FREDERICK STREET
PROPOSED BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

THURBER ENGINEERING LTD.




KEYPLAN

## LEGEND

| | |
|---|---|
| ⯐ | Borehole (Current Investigation) |
| ⯐ | Borehole and Cone (Previous Investigation) |
| ⯐ | Cone Penetration Hole (Previous Investigation) |
| N | Blows /0.3m (Std Pen Test, 475J/blow) |
| CONE | Blows /0.3m (60° Cone, 475J/blow) |
| PH | Pressure, Hydraulic |
| ⊽ | Water Level |
| ⊽ | Head Artesian Water |
| | Piezometer |
| 90% | Rock Quality Designation (RQD) |
| A/R | Auger Refusal |

| NO | ELEVATION | NORTHING | EASTING |
|---|---|---|---|
| BH20-01 | 327.5 | 4 813 653.3 | 226 144.0 |
| BH20-02 | 325.0 | 4 813 695.8 | 226 245.9 |
| 01 | 327.0 | 4 813 643.6 | 226 151.3 |
| 02 | 326.9 | 4 813 673.3 | 226 149.1 |
| 03 | 326.7 | 4 813 648.2 | 226 162.7 |
| 04 | 325.9 | 4 813 673.2 | 226 158.5 |
| 05 | 326.1 | 4 813 653.4 | 226 176.0 |
| 06 | 325.4 | 4 813 681.2 | 226 174.8 |
| 07 | 325.9 | 4 813 660.1 | 226 192.0 |
| 08 | 325.6 | 4 813 687.6 | 226 189.1 |
| 09 | 325.8 | 4 813 666.4 | 226 206.0 |

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
3) Coordinate system is MTM NAD 83 Zone 10.

**GEOCRES No. 40P8-285**

REVISIONS | DATE | BY | DESCRIPTION

DESIGN GL | CHK PKC | CODE | LOAD | DATE FEB 2021
DRAWN MFA | CHK GL | SITE | STRUCT | DWG 1

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# Appendix D
# Foundation Comparison

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## COMPARISON OF FOUNDATION ALTERNATIVES

| Spread Footings | Augured Caissons | Driven H-Piles |
|---|---|---|
| ***Advantages:***<br>i. Generally less costly than deep foundation elements.<br>ii. Relatively high values of geotechnical resistance are available on the very stiff to hard silty clay and very dense sandy silt to silty sand. | ***Advantages:***<br>i. High geotechnical resistance available for caissons extended to till.<br>ii. Construction of caissons could continue in freezing weather.<br>iii. Excavation and dewatering requirements are minimized.<br>iv. Minimal disruption to traffic particularly at the piers since pile caps are not required. | ***Advantages:***<br>i. Higher bearing capacity than spread footings.<br>ii. Minimal excavation and dewatering required.<br>iii. Pile driving could continue in freezing weather.<br>iv. Allows integral abutment design. |
| ***Disadvantages:***<br>i. Larger excavations compared to deep foundations.<br>ii. Construction dewatering will be required. | ***Disadvantages:***<br>i. Higher unit cost compared to other foundation options such as footings or driven piles.<br>ii. Temporary liners and synthetic slurry will be required to install caissons under the water table.<br>iii. Potential difficulty in cleaning and inspecting sockets.<br>iv. Installation through cobbles and boulders will be difficult. | ***Disadvantages:***<br>i. Higher unit costs than footings.<br>ii. Potential for pile deflection or refusal on cobbles, boulders and rock fragments within till.<br>iii. Potential for varying pile lengths within a foundation unit.<br>iv. Will require roadway protection for pile cap construction at piers.<br>v. Vibrations resulting from pile driving activities may impact adjacent structures and utilities, and nearby residents. |
| **FEASIBLE** | **RECOMMENDED** | **FEASIBLE** |

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**Appendix E**
**List of OPSS Documents and NSSP Wording**

## 1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS.PROV 206
- OPSS.PROV 501
- OPSS.PROV 517
- OPSS.PROV 539
- OPSS.PROV 804
- OPSS.PROV 902
- OPSS.PROV 903
- OPSS.PROV 1010
- OPSD 3090.101
- OPSD 3101.150
- OPSD 3102.100
- SP 109F57
- SP 109S12
- SP 517F01
- SP FOUN0003

## 2. Suggested Text for NSSP on Pile Driving

Hard driving conditions through the hard/very dense soils should be expected. Cobbles and boulders should also be anticipated within the silty clay till deposit which may affect pile installation. In order to minimize pile damage while driving the piles hard/dense zones, cobbles, and boulders, to achieve the required tip elevations and soil resistance, it is recommended that the pile tips be reinforced with Titus steel (Standard H-point).

If the piles meet refusal at a depth less than the anticipated depth, the Contractor must terminate driving before the pile is damaged due to over-driving. The Contractor must immediately bring it to the attention of the CA. If the CA cannot resolve the issue, it must be referred to the design team for resolution.

### 3. Suggested Text for NSSP on High-Strain Dynamic Testing

High-strain dynamic testing using pile driving analyzer (PDA) shall be conducted as per NSSP – "HIGH-STRAIN DYNAMIC TESTING, DEEP FOUNDATIONS" to assess ultimate pile capacity and establish set criteria. The dynamic testing shall not be carried out until the piles are within 2 m of the design tip elevation.

The location, sequencing and scheduling of the individual pile testing shall be proposed by the Contractor based on the purpose of the testing, and shall be submitted to the Contract Administrator for approval.

High-strain dynamic testing shall be carried out at the end of initial driving on a minimum of 2 piles per foundation element.

Additional high strain dynamic testing (i.e. restrike testing) shall be carried out during the retapping of piles, as specified in the Retapping Tests on Piles clause. Restrike testing shall be performed on a minimum of 10% of piles in each pile group, rounded up, but no fewer than 2 piles; or as specified in the Contract Documents. Restrike testing shall be carried out no sooner than 24 hours after installation of the individual pile or at a time specified in the Contract Documents. If the hammer needs to be warmed up prior to performing a restrike, it shall not be warmed up by striking the intended test pile.

### 4. Suggested Text for NSSP on "Installation of Caissons"

All caissons shall be installed in accordance with OPSS.PROV 903 and SP 109F57 (April 2018). The caisson installation equipment should be able to dislodge and remove any obstructions such as cobbles and boulders and penetrate the silty clay till.

The caissons will extend below the groundwater table. Soil sloughing and water seepage will occur in unsupported holes primarily within the sand fill, sand, and silty sand/sandy silt layers. Therefore, construction of caissons will require the use of temporary steel liners with synthetic slurry to balance hydrostatic head to support the caisson sidewalls and to provide seepage cut-off where required.

The Contractor shall use appropriate means such as a cleanout bucket, air lift, hydraulic pump, or other devices approved by Engineer to clean the bottom of the excavation of all shafts. A clean-out bucket alone is not sufficient for final clean-out. The cleaning methods, inspection method, and any additional measures required to satisfy the acceptance criteria must be selected by the Contractor to ensure direct contact between the concrete and undisturbed soil. It is the Contractor's responsibility to apply means necessary (such as air lift pump or hydraulic pump, etc.) to clean the socket base and sidewalls.

The bottom of the excavated shaft shall be inspected using a Shaft Inspection Device (SID), Shaft Quantitative Inspection Device (SQUID), down-hole camera, and/or an approved alternate to verify socket cleanliness and thickness of base sediment at the time of concreting. A minimum of 50 percent of the base of each shaft shall have less than 15 mm of sediment at the time of concrete placement. The maximum depth of sediment or any debris at any place on the base of the shaft shall not exceed to 40 mm at the time of concrete placement.

A shaft inspection field report shall be submitted to the Engineer for acceptance prior to proceeding with construction. Concrete placement shall commence no later than 6 hours after acceptance of the excavation. Any accumulated water within the hole may have to be pumped out prior to placing concrete. If accumulated water in the caisson hole cannot be removed, tremie techniques shall be used to place concrete inside the caisson hole.

## 5. Suggested Text for NSSP on "Vibration Monitoring"

Vibrations produced during pile driving and existing structure demolition may disrupt residents and damage nearby structures and utilities. If driven piles are chosen as the foundation option, vibration monitoring is recommended during pile driving to limit potential impacts on existing facilities and residents, and conditions carefully monitored for signs of disturbance. A preconstruction condition survey of existing structures and utilities should be carried out prior to commencement of pile installation.

It is understood that the City of Kitchener does not provide limits on vibration levels. Therefore, it is recommended that the vibration levels stipulated in the City of Toronto By-law 514-2008 be adopted for this project. The limits are provided in the table below.

| Vibration Frequency (Hz) | Vibration Peak Particle Velocity (mm/s) |
|---|---|
| Less than 4 | 8 |
| 4 to 10 | 15 |
| More than 10 | 25 |

## 6. Suggested Text for NSSP on "Groundwater Control"

Water seepage into the temporary excavations from the fill and native soils, and from surface runoff and precipitation, should be expected. Excavation of the cohesionless native soils below the groundwater level without prior dewatering is not recommended since the inflow of groundwater will cause boiling and sloughing of the soil below the water table making it difficult to maintain a dry, sound base on which to work. Suitable systems that might be considered to maintain an unwatered condition at this site include pumping from filtered sumps for nominal penetration below the groundwater level, sheeted excavation (cofferdam) or vacuum well-points for deeper excavations. The dewatering systems must be installed and made operational prior to excavating below the groundwater level and must be effective to lower the groundwater level at least 0.5 m below the footing/pile cap grade level to avoid base boiling in the native soils.

Dewatering of all excavations should be carried out in accordance with OPSS. PROV 517, SP 517F01 Amendment to OPSS 517, November 2016 (issued July 2017), NSP FOUN0003 and OPSS. PROV 902 and SP 109S12. It is recommended that a Professional Engineer with greater than 5 years of experience in designing dewatering systems be retained by the Contractor.

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# Appendix F
# Site Photographs


**Photograph 1 – Frederick Street Underpass, East Abutment (looking Northeast)**

**Photograph 2 – Frederick Street Underpass, West Abutment (Looking Northwest)**