THURBER ENGINEERING LTD.

FOUNDATION INVESTIGATION AND DESIGN REPORT
METROLINX RAILWAY BRIDGE FROM WELLINGTON STREET NORTH TO VICTORIA STREET CONNECTION
HIGHWAY 7-NEW, KITCHENER TO GUELPH
G.W.P. 408-88-00

GEOCRES No. 40P8-277

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Report

to

WSP

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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 Metrolinx bridge/tracks over the planned Wellington Street North to Victoria Street Connection in the Regional Municipality of Waterloo, Ontario. The proposed structure is part of the Highway 7-New Project.

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, stratigraphic profiles, 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 a previous foundation report prepared for this site during the preliminary design phase. The title of the report is:

- Preliminary, Foundation Investigation and Design Report, Bruce Street Extension Under CNR, Highway 7-New, Kitchener to Guelph, G.W.P. 408-88-00, Geocres No. 40P8-163, Report to Ministry of Transportation Ontario West Region, File: 15-64-17, dated June 2, 2009. (Reference 1).

2. SITE DESCRIPTION

The site lies approximately 100 m east of the Kitchener-Waterloo Expressway (KWE) and the existing Metrolinx Rail bridge, and 115 m north of Victoria Street, in the Regional Municipality of Waterloo, Ontario. Twin Metrolinx tracks run from east to west at this site. The rail tracks are built within an embankment that is approximately 3.0 m high.

The site lies within an area of industrial and commercial lands and is generally flat. A parking lot is currently situated on the south side of the Metrolinx tracks. The lands immediately north of the Metrolinx tracks are vacant and covered with long grass and shrubs.

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.

3. INVESTIGATION PROCEDURES

A detailed geotechnical investigation was conducted from July 2 to 19, 2019 and consisted of drilling four boreholes (numbered CN16-13 to CN16-16) at the proposed bridge abutments. Boreholes CN16-14 and CN16-15 were terminated at 35.2 m depth (Elevations 288.0 and 286.9), and Boreholes CN16-13 and CN16-16 at 15.8 m depth (Elevations 307.1 and 306.3). It should be noted that no borehole was drilled to investigate the railway embankment due to access constraints as well as restrictions imposed by Metrolinx.

The approximate locations of the present boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix D. The coordinates and elevations of the boreholes drilled during the present investigation are given on the drawings and on the individual Record of Borehole Sheets in Appendix A.

Borehole 08-046, drilled during the previous investigation (Reference 1), has been incorporated in this report. Borehole 08-046 was terminated at 33.7 m depth (Elevation 288.2). The Record of Borehole sheet of Borehole 08-046 is included in Appendix B.

The ground surface elevations and coordinates of the recent boreholes were provided by WSP.

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

During the present investigation, the boreholes were drilled using a track-mounted drill rig and advanced with a combination of hollow stem augers and mud rotary drilling. Samples were obtained at selected depth intervals using a split spoon sampler in conjunction with 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 present investigation are presented on the Record of Borehole sheets in Appendix A.

Groundwater conditions in the open boreholes were observed during the drilling operations. One piezometer was installed in Borehole CN16-16 to permit long term monitoring of groundwater levels. The piezometer consisted of 25 mm diameter PVC pipe with a slotted screen enclosed in filter sand. The location and completion details of the piezometer are summarized in Table 3.1 along with the borehole completion details. The completion of the boreholes was carried out in accordance with the requirements of O. Reg. 903 (as amended by O. Reg. 372/07). The piezometers are planned to be decommissioned in the summer of 2020.

Table 3.1 – Borehole Completion Details

Foundation Unit	Borehole	Borehole Ground Surface Elevation	Borehole Depth / Base Elevation (m)	Piezometer Tip Depth / Elevation (m)	Completion Details
West Abutment	CN16-13	322.9	15.8/307.1	None installed	Borehole backfilled with cement and grout to 0.6 m, then bentonite holeplug to surface.
	CN16-15	322.1	35.2/286.9	None installed	Borehole backfilled with bentonite holeplug, sand and cement to surface.
	CN16-16	322.2	15.8/306.3	7.6/314.6	Piezometer with 3.0 m slotted screen installed with grout from 15.8 m to 7.6 m, sand filter from 7.6 m to 4.0 m, bentonite holeplug from 4.0 m to ground surface.
East Abutment	CN16-14	323.2	35.2/288.0	None installed	Borehole backfilled with cement, gravel and bentonite holeplug to surface.
	08-046	321.9	33.7/288.2	33.5/288.4	Piezometer with 1.5 m slotted screen installed with sand filter to 31.7 m, holeplug to 30.8 m, grout mix with auger cuttings to 1.2 m, holeplug to 0.6 m, then concrete to surface.

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 are summarized on the Record of Borehole sheets and presented on the figures included in Appendix A.

The results of the laboratory testing conducted during the previous investigation (Reference 1) are summarized on the Record of Borehole sheets in Appendix B, and also presented on the figures included in Appendix B.

In order to assess the potential for sulphate attack on concrete foundations, as well as the potential for corrosion associated with the structure, a sample of the existing native soil was collected. The sample was submitted to SGS Canada Inc., a CALA accredited analytical laboratory in Lakefield, Ontario, for analytical testing of corrosivity parameters and sulphate

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content. The results of the analytical testing are summarized in Section 6 and are presented in Appendix C.

5. DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendices A and B. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following paragraphs. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description and must be used for interpretation of the site conditions. It should be recognized and expected that soil conditions may vary between and beyond borehole locations.

In general, the soil stratigraphy at this site consisted of surficial topsoil over loose to dense silty sand to sand fill, overlying native compact to dense silty sand, sandy silt and upper sand and compact gravelly sand. An extensive deposit of very stiff to hard silty clay was contacted below the cohesionless soils. Underlying the silty clay, layers of very dense sand and sandy silt till were contacted. Layers of very dense sandy silt till/sand and silt till and very stiff to hard clayey silt till were contacted within the silty clay deposit. A layer of very dense sand and gravel was contacted below the sandy silt till in Borehole CN16-15. The groundwater level measured in the piezometers was 1.9 m below the ground surface (Elevation 320.3).

5.1 Topsoil

Topsoil was encountered surficially in the four boreholes drilled at this site during the present investigation. The thickness of the topsoil layer ranged from 100 mm to 150 mm.

The topsoil thickness may vary between the borehole locations and in other areas of the site.

5.2 Cohesionless Fill

Cohesionless fill was encountered immediately below the topsoil in Boreholes CN16-13 to CN16-16 drilled at the site during the present investigation, and surficially in Borehole 08-046 drilled during the previous investigation.

The cohesionless fill consisted of brown to black sand and silty sand containing trace gravel to gravelly, trace to some clay and occasional cobbles. Occasional organics and decayed wood fragments were encountered in the fill in Boreholes CN16-13, CN16-15, CN16-16 and 08-046. A gasoline odour was noted in the cohesionless fill in Boreholes CN16-16 and 08-046.

The thickness of the cohesionless fill ranged from 2.1 m to 2.9 m, with the lower boundary of this layer encountered at depths ranging from 2.1 m to 3.0 m (Elevation 320.2 to 319.1).

SPT N-values recorded in the cohesionless fill generally ranged from 5 to 34 blows for 0.3 m penetration, indicating a loose to dense state. SPT 'N' values of 1 to 3 blows per 0.3 m of penetration, indicating a very loose state, were measured in Borehole CN16-15 and 08-46. Moisture content of samples of the cohesionless fill generally ranged from 6 percent to 30 percent. A moisture content of 98 percent was measured within the cohesionless fill in Borehole CN16-15, at a depth of 2.6 m, indicating the presence of organics.

The results of grain size analyses conducted on samples of the silty sand and sand fill are provided on the Record of Borehole sheets in Appendix A, and illustrated on Figure A1 of Appendix A. The results are summarized as follows:

Soil Particles	Cohesionless Fill (Percent)
Gravel	4 to 22
Sand	37 to 75
Silt	17 to 32
Clay	3 to 11

5.3 Organics

A layer of organics was encountered below the cohesionless fill layer in Borehole CN16-14, at a depth of 3.0 m (Elevation 320.2). The thickness of the organics layer was 500 mm.

The SPT N-Value recorded in the organic layer was 15 blows for 0.3 m penetration, indicating a compact state.

The moisture content from a sample of the organics layer was measured to be 124 percent.

5.4 Silty Sand, Sandy Silt and Upper Sand

Native brown to grey silty sand to sandy silt was encountered below the cohesionless fill and organics at depths ranging from 2.1 m to 3.5 m (Elevations 319.9 to 319.1) in Boreholes CN16-13, CN16-14, CN16-16 and 08-046. The thickness of the silty sand/sandy silt ranged from 1.1 m to 4.3 m.

An upper layer of brown sand containing trace silt and trace clay was contacted at 3.0 m depth (Elevation 319.1) in Borehole CN16-15. The thickness of the sand was 4.2 m.

Occasional cobbles were encountered within the sandy silt layer in Borehole 08-046. A gasoline odour was also noted in the sandy silt layer in Borehole 08-046.

SPT N-values recorded in the silty sand, sandy silt and upper sand ranged from 11 to 41 blows for 0.3 m penetration, indicating a compact to dense state. An SPT 'N' value of 7 blows per 0.3 m of penetration, indicating a loose condition, was measured in Borehole 08-046 near Elevation 319.5.

Moisture content of samples of the silty sand, sandy silt and upper sand generally ranged from 12 percent to 23 percent. A moisture content of 46 percent was measured below the layer of organics in Borehole CN16-14.

Samples of the sand and sandy silt from the present and previous investigation, underwent laboratory gradation analysis. These results are summarized on the Record of Borehole sheets included in Appendices A and B. The grain size distribution curves for these samples are plotted on Figures A2 and B1 of Appendices A and B. The results of this testing are summarized as follows:

Soil Particles	Sandy Silt (Percent)	Sand (Percent)
Gravel	2	0
Sand	44	91
Silt	50	8
Clay	4	1

5.5 Gravelly Sand

A layer of brown gravelly sand containing trace silt and trace clay was encountered below the native silty sand layer in Borehole CN16-16, at a depth of 5.6 m (Elevation 316.6). The thickness of the gravelly sand layer was 1.6 m.

The depth to the base of the gravelly sand was contacted at 7.2 m (Elevation 315.0).

An SPT N-value recorded in the gravelly sand was 15 blows for 0.3 m penetration, indicating a compact state.

The moisture content of a sample of the gravelly sand was 13 percent.

5.6 Clayey Silt Till

Brown to grey clayey silt till with sand to some sand, trace gravel and occasional silty sand seams was encountered below the sandy silt and sand layers in Boreholes CN16-15 and 08-046, at depths of 7.2 m and 6.4 m, respectively (Elevations 314.9 and 315.5), respectively.

The thickness of the clayey silt till deposit ranged from 2.7 m to 4.3 m, with the lower boundary encountered at depths of 11.5 and 9.1 m (Elevations 310.6 and 312.8) in Boreholes 16-15 and 08-046, respectively.

SPT N-values recorded in the clayey silt till ranged from 23 blows to 45 blows for 0.3 m penetration, indicating a very stiff to hard consistency. Moisture content of samples of the clayey silt till generally ranged from 10 percent to 26 percent.

The results of grain size distribution analyses carried out on samples of the clayey silt till are presented on the Record of Borehole sheets included in Appendices A and B. Grain size distribution curves of the samples tested are presented on Figures A3 and B2 of Appendices A and B. The results of the grain size distribution analyses are summarized below:

Soil Particles	Clayey Silt Till (Percent)
Gravel	1 to 4
Sand	18 to 25
Silt	46 to 54
Clay	25 to 27

The results of Atterberg Limits tests conducted on samples of clayey silt till are presented on the Record of Borehole sheets in Appendices A and B, and illustrated in Figures A6 and B6 of Appendices A and B. The results are summarized as follows:

Index Property	Percentage (%)
Liquid Limit	24 to 26
Plastic Limit	14
Plasticity Index	10 to 13

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

It should be noted that glacial tills are known to contain cobbles and boulders.

5.7 Silty Clay

Brown to grey silty clay containing trace sand was encountered below the silty sand, sandy silt and sand layers in Boreholes CN16-13 and CN16-14, below clayey silt till layer in Boreholes CN16-15 and 08-046, and below the gravelly sand layer in Borehole CN16-16. The silty clay was contacted at depths ranging from 4.1 m to 11.5 m (Elevations 318.8 to 310.6).

Where fully penetrated in Boreholes CN16-14, CN16-15 and 08-046, the silty clay layer ranged in thickness from 15.9 m to 23.6 m, with the base of the layer contacted at depths from 27.4 m to 30.8 m (Elevation 294.7 to 292.4). Boreholes CN16-13 and CN16-16 were terminated in the silty clay layer at a depth of 15.8 m (Elevations 307.1 and 306.3). Layers of sandy silt till and sand and silt till were encountered within the silty clay in Borehole CN16-14.

SPT N-values recorded in the silty clay ranged from 18 to 90 blows for 0.3 m penetration, indicating a very stiff to hard consistency. SPT ‘N’ values of 100 blows for less than 0.3 m penetration, indicating a hard consistency, were also measured in Boreholes CN16-15 and 08-046 near Elevations 302.0 and 305.0.

Moisture content of samples of the silty clay generally ranged from 12 percent to 38 percent.

The results of grain size distribution analyses carried out on samples of the silty clay are presented on the Record of Borehole sheets included in Appendices A and B. Grain size distribution curves of the samples tested are presented on Figures A4 and B3 of Appendices A and B. The results of the grain size distribution analyses are summarized below:

Soil Particles	Silty Clay (Percent)
Gravel	0
Sand	1 to 7
Silt	23 to 45
Clay	54 to 75

The results of Atterberg Limits tests conducted on samples of silty clay are presented on the Record of Borehole sheets in Appendices A and B, and illustrated in Figures A7 and B7 of Appendices A and B. The results are summarized as follows:

Index Property	Percentage (%)
Liquid Limit	39 to 50
Plastic Limit	17 to 21
Plasticity Index	21 to 30

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

5.8 Lower Sand

A lower sand layer was encountered below the silty clay in Boreholes CN16-15 and 08-046, at depths of 27.4 m and 28.7 m (Elevation 294.7 and 293.3), respectively. The lower sand was generally brown to grey in colour, with some silt, trace to some gravel and trace clay.

The thickness of the lower sand layer ranged from 1.5 m to 3.7 m, with the bottom boundary encountered at depths of 31.1 m and 30.2 m (Elevations 291.0 and 291.8) in Boreholes CN16-15 and 08-046, respectively.

SPT N-values recorded in the lower sand ranged from 79 to 80 blows for 0.3 m penetration, indicating a very dense relative density.

Moisture content of samples of the lower sand generally ranged from 12 percent to 20 percent.

One sample of the lower sand underwent laboratory gradation analysis. These results are presented on the Record of Borehole sheets included in Appendix B and the grain size distribution curves for these samples are plotted on Figure B4 of Appendix B. The results of this testing are summarized as follows:

Soil Particles	Lower Sand (Percent)
Gravel	5
Sand	69
Silt and Clay	26

5.9 Sandy Silt Till and Sand and Silt Till

Layers of grey sandy silt till and, sand and silt till containing trace clay and occasional cobbles were contacted at 11.7 m and 14.7 m depth (Elevations 311.2 and 308.5) in Boreholes CN16-13 and CN16-14, and also at depths ranging from 30.2 m and 31.1 m (Elevations 291.0 to 292.4) in Boreholes CN16-14, CN16-15 and 08-46. Where fully penetrated, the thickness of the sandy silt till/sand and silt till ranged from 1.8 m to 3.3 m.

The depth to the base of the upper sandy silt till and, sand and silt till layers in Boreholes CN16-13 and CN16-14 was at 14.8 m and 16.5 m (Elevations 308.1 and 306.7). The depth to the base of the lower sandy silt till layer in Borehole CN16-15 was 34.4 m (Elevation 287.7).

Boreholes 08-046 and CN16-14 were terminated in the sandy silt till layer at depths of 33.7 m and 35.2 m, respectively (Elevation 288.2 and 288.0).

SPT N-values recorded in the sandy silt till/sand and silt till ranged from 76 blows for 0.3 m penetration to 100 blows for 0.05 m penetration, indicating a very dense relative density. Tricone grinding was noted in this layer, which indicates the presence of cobbles and/or boulders.

Moisture content measured in the sandy silt till/sand and silt till generally ranged from 7 percent to 27 percent.

The results of grain size distribution analyses carried out on samples of the sandy silt till/sand and silt till are presented on the Record of Borehole sheets included in Appendices A and B. Grain size distribution curves of the samples tested are presented on Figures A5 and B5 of Appendices A and B. The results of the grain size distribution analyses are summarized below:

Soil Particles	Sandy Silt Till/ Sand and Silt Till (Percent)
Gravel	0 to 4
Sand	31 to 44
Silt	43 to 61
Clay	7 to 19

It should be noted that glacial tills are known to contain cobbles and boulders.

5.10 Sand and Gravel

A layer of grey sand and gravel was encountered below the sandy silt till in Borehole CN16-15, at a depth of 34.4 m (Elevation 287.7).

Borehole CN16-15 was terminated within the sand and gravel layer at a depth of 35.2 m (Elevation 286.9).

The SPT N-value recorded in the gravelly sand was 100 blows for 0.1 m penetration, indicating a very dense condition. The moisture content of the sand and gravel was 13 percent.

5.11 Groundwater Conditions

Water levels were observed in the boreholes during and upon completion of drilling. Standpipe piezometers were installed in Boreholes 08-046 and CN16-16, to monitor water levels after completion of drilling. The water levels measured in the piezometer installed at CN16-16 are summarized in Table 5.1, along with the measurements in the open boreholes upon completion of drilling. Unfortunately, the piezometer installed at borehole 08-046 was destroyed before any reading could be obtained.

Previous geotechnical investigation conducted in 1966 (information provided in Reference 1), indicates that groundwater level is near Elevation 318.4.

Water level was measured at 3.2 m depth (Elevation 319.0) on October 5, 2008, in a previous piezometer installed at the site, in close proximity to the existing CN bridge over KWE.

Table 5.1 – Water Level Measurements

Borehole	Date	Water Level (m)		Comment
		Depth	Elevation	
CN16-14	July 17, 2019	4.6	318.6	Open borehole
CN16-16	July 2, 2019 Aug 8, 2019 Aug 29, 2019	4.0 1.9 1.9	318.2 320.3 320.3	Open borehole Piezometer Piezometer
08-046	Aug 7, 2008	N/A	N/A	No water level readings available - piezometer was destroyed.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. The groundwater levels may be at a higher elevation after periods of significant or prolonged precipitation.

6. CORROSIVITY AND SULPHATE TEST RESULTS

A sample of the silty sand fill from Borehole CN16-15 was 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 C.

Table 6.1 – Analytical Test Results

Parameter	Units (Soil)	Test Results
		CN16-15, SS4 Depth 2.6 m
Soil Redox Potential	mV	255
Sulphide	%	0.02
pH	pH Units	7.88
Chloride	µg/g	60
Sulphate	µg/g	100
Conductivity	uS/cm	400
Resistivity (calculated)	ohms.cm	2500

7. MISCELLANEOUS

Landshark Drilling of Brantford, Ontario supplied a rubber track mounted B-57 drill rig and conducted the drilling, sampling and in-situ testing operations for the present investigation.

The coordinates for the boreholes were obtained with GPS equipment by Thurber, and the elevations were provided by WSP.

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 for the present investigation was conducted by Dr. Nancy Berg, P.Eng.

Interpretation of the data and preparation of the current report was carried out by Ms. Judy Mei, EIT and Ms. Rocio Palomeque Reyna, 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.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

8. GENERAL

This report presents an 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 a new structure to carry the Metrolinx tracks over the proposed Wellington Street North to Victoria Street Connection road to be located east of the KWE and the existing Metrolinx bridge in the Regional Municipality of Waterloo, Ontario.

The General Arrangement (GA) drawing provided by WSP, dated November 2018, indicates that the new Metrolinx bridge over Wellington Street to Victoria Street Connection will be a single span rigid frame structure supported by two abutments with proposed strut beams connecting the base of the abutments. The proposed length of the structure is 18.2 m, and the width is 10.0 m. The new Metrolinx Railway Wellington Street to Victoria Street Connection road will be constructed in a cut up to 5.8 m deep, and the final grade within the zone of the proposed Metrolinx bridge will be at approximate Elevation 318.5. Metrolinx tracks, within the structure limits, will be maintained at the existing Elevation of 326.0.

Subject to discussions with Metrolinx, construction of the structure will likely have to be done in stages in order to keep at least one track operation. Track protection will be required for this stage of construction.

It is understood that another new Metrolinx bridge will be constructed approximately 30 to 40 m west of this site, to accommodate the proposed S-E Ramp which will pass under the existing twin Metrolinx tracks.

This foundation investigation and design report, with the interpretation and recommendations, is 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 information provided by WSP/MTO and on the factual data obtained in the course of this investigation.

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, ψ, of 1.0 has been used for assessing ULS and SLS factored geotechnical resistances. Should the consequence classification changes, the geotechnical assessment and recommendations will need to be reviewed and revised as necessary. Since the bridge will be used to carry rail tracks, foundation recommendations have also considered AREMA guidelines.

10. STRUCTURE FOUNDATION

The stratigraphy identified in the geotechnical investigation consisted primarily of topsoil over loose to dense silty sand to sand fill, overlying native compact to dense silty sand, sandy silt and upper sand and compact gravelly sand. An extensive deposit of very stiff to hard silty clay was contacted below the cohesionless soils. Underlying the silty clay, layers of very dense sand and sandy silt till were contacted. Layers of very dense sandy silt till/sand and silt till and very stiff to hard clayey silt till were contacted within the silty clay deposit. A layer of very dense sand and gravel was contacted below the sandy silt till in Borehole CN16-15. The groundwater level measured in the piezometers was 1.9 m below the ground surface (Elevation 320.3).

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

1. Spread footings bearing on native soil
2. Spread footings on engineered fill
3. Augered caissons (drilled shafts)
4. Steel H-piles or steel pipe driven into the very dense glacial till soils

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

10.1 Spread Footing on Native Soil

Spread footings bearing on native soil generally are a cost-effective form of foundation and are feasible at this site, however approximately 10 to 11 m deep temporary excavations will be required to construct the footings. According to the GA drawing, the proposed base of the abutment footing is at approximately Elev. 316.0.

The existing fill is not considered suitable for the support of spread footings, and the spread footings should bear on native undisturbed compact to dense sand/sandy silt and very stiff to hard silty clay. Provided a minimum footing width of 2 m is maintained, the spread footings may be designed in accordance with the elevations and bearing resistances given in Table 10.1.

Table 10.1 – Geotechnical Resistances for Spread Footings

Foundation Unit	Borehole	Approximate Highest Founding Elevation (m)	Founding Stratum	Factored ULSf (kPa)	Factored SLS_f (up to 25 mm settlement) (kPa)
West Abutment	CN16-13 CN16-15 CN16-16	315.0	Very stiff silty clay	400	300
East Abutment	CN16-14 08-046	315.0	Very stiff silty clay	400	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 factored Geotechnical Resistance at SLS was assessed assuming a factor of 0.8 for typical degree of understanding of the subsurface conditions.

Based on AREMA guidelines, an allowable bearing capacity of 300 kPa may be used for footing design.

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, as well as the allowable bearing capacity value, given above are based on an estimated total settlement not exceeding 25 mm. This 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 placed on the native, undisturbed soils may be computed based on an ultimate coefficient of friction, tan δ, 0.35 for the very stiff silty clay. A resistance Factor of 0.6 should be applied for cohesive soils, as indicated in Table 6.2 in the CHBDC (2019).

The groundwater level measured in the piezometer was 1.9 m below the ground surface (Elevation 320.3). Founding elevation presented in Table 10.1 will be below groundwater level observed during the investigation. Local groundwater control, as discussed in Section 17, will be required to construct the footing in the dry and to prevent disturbance and base heave of the footing base.

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 concrete of the same strength and class as that of the footing. Where sub-excavation is required to remove unsuitable material from below the design founding level, the founding surface should be re-established using the same concrete.

10.2 Spread Footing on Engineered Fill

Spread footings can also be founded on Granular "A" engineered fill pads, where this is beneficial to the overall design. However, this option will also involve approximately 10 to 11 m deep temporary excavation to construct the engineering fill pad.

If an engineered fill pad is used, all topsoil, organics or other deleterious materials must be stripped from the footprint of the foundation to expose competent native subgrade material. Subexcavation of existing surficial fill soils will be required. The engineered fill will bear on native very stiff silty clay or clayey silt till and the highest permitted founding/base elevation at which engineered fill pads may be placed, is given in Table 10.2.

Table 10.2 – Highest Founding Elevations for Engineered Fill Pads

Foundation Unit	Borehole	Highest Founding Elevation (m)
West Abutment	CN16-13 CN16-15 CN16-16	316.0
East Abutment	CN16-14 08-046	316.0

Provided a minimum footing width of 2 m is maintained footings bearing on the well compacted engineered fill pad, at least 2-m thick, may be designed for the following geotechnical resistances:

Factored Geotechnical Resistance at ULS	900 kPa
Factored Geotechnical Resistance at SLS	350 kPa

These resistance values are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC Clauses 6.10.2 to Clause 6.10.5.

Based on AREMA guidelines, an allowable bearing capacity of 350 kPa may be used for footing design.

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 Factored Geotechnical Resistance at SLS was assessed assuming a factor of 0.8 for typical degree of understanding of the subsurface conditions.

Temporary excavations required to construct the engineered fill pad will extend below the water table. Local groundwater control, as discussed in Section 17, will be required to construct the engineered fill pad in the dry and to prevent disturbance of the engineered fill pad base.

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is expected to not exceed 25 mm. Differential settlements are not expected to exceed 20 mm across the width of the structure.

The sliding resistance of cast-in-place concrete placed on the engineered fill may be computed based on an ultimate coefficient of friction, tan δ, of 0.55. A resistance Factor of 0.8 should be applied for cohesionless soils, as indicated in Table 6.2 in the CHBDC (2019).

The bases of the foundation excavations should be inspected by a geotechnical engineer to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to place the engineered fill. The Granular A for the engineered fill pad must be compacted to 100% Standard proctor maximum dry density (SPMDD) at optimum moisture content ±2%, and placed in 300 mm lifts. The geometry of the fill pad must conform to the general requirements shown in Figure 1 in Appendix E.

10.3 Augered Caissons (Drilled Shafts)

Drilled shaft foundations founded on very dense sandy silt till were considered for the support of structural loads at this site. However, augered caissons (drilled shafts) are not recommended for use as foundation support at this site, due to the depth to suitable bearing material, greater than 20 m, and potential caisson installation difficulties including basal boiling and heave due to the presence of water bearing sand and sandy silt till deposit below the silty clay layer. Sealing of the caisson liner into the founding stratum may be difficult.

10.4 Steel H-Piles and Steel Pipe Piles

From a foundation engineering perspective, it is feasible to support the structure on steel H-piles driven to practical refusal in the very dense sandy silt till. Open ended steel pipe piles may also be considered as an alternate foundation option. It should be noted that pipe piles driven into very dense sandy silt till deposit are more prone to pile tip damage in comparison to H-piles.

It is recommended that the H-piles be driven to achieve resistance in the very dense sandy silt till encountered at this site.

10.4.1 Axial Resistance

The axial resistances of HP 310 X 110 and HP 360 x 132 steel piles, and 324 mm diameter and 356 mm diameter steel piles driven to refusal in very dense cohesionless till were assessed based on the subsurface conditions encountered at the abutment locations. The estimated Ultimate Limit States (ULS) and geotechnical resistance at Serviceability Limit States (SLS), as well as the recommended pile tip elevations are summarized in Tables 10.3 and 10.4.

Table 10.3 – Estimated Axial Resistance and Pile Tip Elevation for H-Piles

Foundation Unit	Borehole	Approx. Pile Tip Elevation (m)	Minimum Pile Length Assumed (m)	Plie Section HP 310 X 110		Pile Section HP 360 X 132	
				Factored ULS (kN)	Factored SLS_f (kN)	Factored ULS (kN)	Factored SLS_f (kN)
West Abutment	CN16-15	289.0	27.0	1,500	1,300	1,650	1,450
East Abutment	CN16-14 08-046	290.0	26.0	1,500	1,300	1,650	1,450

Table 10.4 – Estimated Axial Resistance and Pile Tip Elevation for pipe piles

Foundation Unit	Borehole	Approx. Pile Tip Elevation (m)	Minimum Pile Length Assumed (m)	Plie Section 324 mm diameter Wall Thickness 12.7 mm		Pile Section 356 mm diameter Wall Thickness 12.7 mm	
				Factored ULS (kN)	Factored SLS_f (kN)	Factored ULS (kN)	Factored SLS_f (kN)
West Abutment	CN16-15	289.0	27.0	1,200	1,050	1,400	1,200
East Abutment	CN16-14 08-046	290.0	26.0	1,200	1,050	1,400	1,200

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.

Based on AREMA guidelines, allowable bearing capacity values equivalent to the above SLS values for respective pile types may be used for pile design.

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

10.4.2 Downdrag

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

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10.4.3 Lateral Resistance

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

Silty Clay (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 pile, kPa
	C_u	=	undrained shear strength of cohesive soils, kPa
	γ	=	unit weight of soil, kN/m^3
	B	=	width of pile, m

Silty Sand, Sandy Silt, Sand, Sandy Silt Till (cohesionless soils)

	k_s	=	$n_h . z / B$ (kN/m^3)
	p_{ult}	=	$3 . \gamma' . z . K_p$ (kPa)
where	z	=	depth of embedment of pile, m
	B	=	pile width, m
	n_h	=	coefficient related to soil density, kN/m^3 , Table 10.5
	γ'	=	Bouyant unit weight of soil, kN/m^3, Table 10.5
	K_p	=	passive earth pressure coefficient, Table 10.5

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressure obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K, for analysis may be obtained by the expression, $K = k_s \times d_z \times B$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), B is the pile width (m), d_z is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult} \times d_z \times B$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements.

For pile lateral resistance design below the flexible zone, soil-pile interaction analyses may be carried out using the coefficient of horizontal subgrade reaction values provided in Table 10.5 below.

Table 10.5 – Recommended Geotechnical Parameters for Lateral Resistance Design

Location	Reference Boreholes	Approx. Elevation (m)	Undrained Shear Strength C_u (kPa)	Unit Weight γ (kN/m^3)	K_p	n_h (kN/m^3)	Soil Conditions
West Abutment	CN16-15	322.0 to 319.0	-	20	2.9	2,200	Very loose to compact silty sand fill
		319.0 to 315.0	-	11*	3.1	3,500	Dense to compact sand
		315.0 to 310.5	180	10*	-	-	Very stiff to hard clayey silt till
		310.5 to 294.7	200	10*	-	-	Hard silty clay
		294.7 to 291.0	-	11*	3.5	6,000	Very dense sand
		291.0 to 287.0	-	11*	3.7	8,000	Very dense silty sand till/sand and gravel
East Abutment	CN16-14 08-046	323.0 to 319.5	-	20	2.9	2,500	Loose to compact sand fill/organics
		319.5 to 316.0	-	11*	3.1	3,500	Compact to very dense silty sand
		316.0 to 292.5	200	10*	-	-	Hard silty clay
		292.5 to 288.0	-	11*	3.7	8,000	Very dense silty sand till

* Bouyant unit weight below water table

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).

10.4.4 Pile Installation

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

At this site, the piles will have to be driven through hard clay into very dense sand and silt till.

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. 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” where “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 in conjunction with the Hiley tests at this site, to ensure the integrity of the pile and to verify pile ultimate geotechnical resistance. PDA testing should be completed for 10 percent the piles for each foundation element or a minimum of 2 piles tested at each foundation element, whichever is more.

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

Glacially derived soils inherently contain cobbles and boulders. Hard driving conditions through the very hard and dense soils should be expected. In order to minimize pile damage while driving through boulders, cobbles and harder/dense zones to achieve the required tip elevations and soil resistance, it is recommended that the pile tips be reinforced with Titus steel (Standard H-point).

Pile tip protection should be provided for open ended pipe piles.

The Contract Documents must contain a NSSP alerting the Bidders to the presence of cobbles and boulders in the glacial tills. Suggested texts for the NSSP’s are included in Appendix H. The NSSP should contain a requirement to terminate driving before the pile is damaged by overdriving.

10.5 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 between the CSP surround and the pile.

It is recognized that the bridge will probably be constructed in accordance with AREMA and with conventional abutments as per the GA drawing.

10.6 Frost Cover

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

10.7 Recommended Foundation

From a geotechnical perspective, and based on available information, the recommended foundations at this site are the following:

- For integral abutments, it is recommended that the abutments be supported on steel H-piles driven into the very dense sandy silt till.
- For non-integral abutments (e.g. rigid frame structure proposed in the GA drawing), it is recommended that the abutments be supported on spread footings founded on native undisturbed very stiff to hard silty clay or an engineered fill pad.

11. RETAINING WALLS

The GA drawing dated November 2018 indicates that construction of four concrete retaining walls are planned at each corner of the proposed structure to retain the existing railway embankment fill and native soils. The locations and lengths of the proposed retaining walls are presented in Table 11.1. Further details of the retaining walls were not provided.

Table 11.1 – Retaining Wall Details

Location relative to the structure	Borehole	Length (m)
Northeast	CN16-14	15
Northwest	CN16-13	14
Southeast	08-046	14
Southwest	CN16-15 CN16-16	15

To provide an acceptable foundation performance, the retaining walls must be founded on native compact silty sand/sandy silt/sand or very stiff silty clay. The highest recommended base levels for the retaining walls are as presented in Table 11.2.

Table 11.2 – Geotechnical Resistances and Founding Elevations for Retaining Walls

<table>
<tr><th>Retaining Wall Location</th><th>Borehole</th><th>Highest Founding Elevation (m)</th><th>Founding Statum</th><th>Factored ULS$_f$ (kPa)</th><th>Factored SLS$_f$ (up to 25 mm settlement) (kPa)</th></tr>
<tr><td>Northeast</td><td>CN16-14</td><td rowspan="3">319.5 (*)</td><td>Compact to dense silty sand</td><td rowspan="3">350 (*)</td><td rowspan="3">250 (*)</td></tr>
<tr><td>Northwest</td><td>CN16-13</td><td>Very Stiff silty clay</td></tr>
<tr><td>Southeast</td><td>08-046</td><td>Compact sandy silt</td></tr>
<tr><td>Southwest</td><td>CN16-15
CN16-16</td><td>319.0 (*)</td><td>Compact silty sand/ sand</td><td>300 (*)</td><td>200 (*)</td></tr>
</table>

Note (*): Higher geotechnical resistance values are available for the retaining wall foundations if founding elevation is lower; recommendations provided in Section 10.1 and 10.2 are applicable for retaining walls.

The geotechnical resistances provided above are for concentric, vertical loading. The effects of load inclination and eccentricity need to be taken into account according to 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.

Based on AREMA guidelines, allowable bearing capacity values equivalent to the above factored SLS values should be used for retaining wall foundation design for the North and South retaining walls respectably.

The sliding resistance of cast-in-place concrete placed on the native, undisturbed soils may be computed based on an ultimate coefficient of friction, tan δ, 0.35 for the very stiff silty clay and 0.45 for the compact silty sand, sandy silt and sand. A resistance Factor of 0.6 should be applied for cohesive soils and, 0.8 for cohesionless soils, as indicated in Table 6.2 in the CHBDC (2019).

A 500-mm thick layer of organics was encountered at 3.0 m depth (Elevation 320.2) in Borehole CN16-14. This layer must be removed within the retaining wall footprint before construction of the retaining wall foundations.

If required, the retaining wall may be founded on engineered fill founded on the compact to dense silty sand/sandy silt/sand/sand/gravelly sand and hard silty clay. Engineered fill placed under the retaining wall to achieve the design founding level must consist of OPSS Granular "A" compacted to 100% of its SPMDD at a moisture content within 2% of optimum.

The sliding resistance of cast-in-place concrete placed on the engineered fill may be computed based on an ultimate coefficient of friction, tan δ, of 0.55. A resistance Factor of 0.8 should be applied for cohesionless soils, as indicated in Table 6.2 in the CHBDC (2019).

Topsoil, organics, loose fill, and any soft/wet material must be stripped from the footprint of the retaining wall. The subgrade under the retaining wall foundation should be inspected and any soft/loose spots should be sub-excavated and replaced with compacted granular materials prior to placing fill. The subgrade preparation for the retaining wall and placement and compaction of the granular fill must be carried out in the dry. Dewatering may be required to prepare the founding base.

Lateral earth pressures acting on the walls should be computed as described in Section 12. If the wall is retaining sloping backfill, appropriate earth pressure parameters for sloping backfill should be used.

The concrete retaining walls must be designed in accordance with American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering and METROLINX General Guidelines for Design of Railway Bridges and Structures (November 2018). These guidelines are adapted from CN Engineering Guidelines for Design of Railway Structures as per the agreement between METROLINX and CN on March 28, 2013.

11.1 Slope Stability of the Retaining Walls

Preliminary analysis of the global stability was conducted to assess stability of retaining walls founded on compact silty sand/sandy silt/sand and very stiff silty clay.

The global stability of the retaining walls must be revisited if the final locations and/or detail configurations of the walls are changed.

Global stability analyses were carried out for the retaining walls utilizing the commercially available slope stability analysis program Slope/W (Version 2019) of the GeoStudio software package developed by Geo-Slope International with the option for Morgenstern-Price method of slices for the limit equilibrium analyses. Analyses were completed for both static and seismic loading conditions.

The soil parameters used in the analyses were estimated from empirical correlations using the results of the in situ Standard Penetration Tests (SPTs) and geotechnical laboratory testing. The groundwater level in our analysis was based on readings obtained from standpipe piezometer.

The stability of the embankment was also checked under seismic loading assuming an acceleration of 0.097 g.

Results of the stability analyses are presented on Figures G1 to G3 in Appendix G. The results are also summarized in Table 11.3 below.

Table 11.3 - Computed Factors of Safety

Condition	Factor of Safety	Figure (Appendix G)
Retaining wall		
Static Drained	1.8	G1
Static Undrained	1.9	G2
Seismic = 0.097 g	1.6	G3

As per typical MTO requirements, a Factor of Safety (F.S.) of 1.3 is acceptable for short term conditions and for total stress (undrained) conditions. A F.S. of 1.5 is acceptable for long term (drained) conditions. Under the assumed seismic loading, the minimum acceptable factor of safety is 1.1. In the case of static loading, the factors of safety against global failure was 1.8 for drained conditions and 1.9 for undrained conditions. Under the estimated seismic loading, the minimum factor of safety calculated was 1.6. These factors of safety are considered to be acceptable for the proposed embankment bearing on the soils encountered at this site.

11.2 Settlement of the Retaining Walls

The construction of the retaining walls, with heights of 7.0 m with new granular backfill behind the walls will induce immediate (elastic) settlement in the underlying compact to dense sand/sandy silt/silty sand and very stiff to hard silty clay.

The immediate settlements were assessed using elastic methods. Based on these analyses, the settlement is estimated to be less than 25 mm. This settlement will be immediate and essentially complete when construction of the retaining wall is completed.

Inspection of the retaining walls and placing of additional granular material to re-establish grades as necessary should be implemented during and after construction.

12. LATERAL EARTH PRESSURES

Earth pressures acting on a structure (e.g. abutment or retaining wall), 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 12.1)
γ = unit weight of retained soil (see Table 12.1)
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 12.1.

Table 12.1 – 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	-

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), 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) might be preferred as it results in lower earth pressures acting on the wall.

The factors in Table 12.1 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 3102.100 where appropriate.

13. NORTH/SOUTH APPROACH - PERMANENT CUT

Permanent earth cuts are required near and through the existing Railway embankment to construct the Wellington Street North to Victoria Street Connection at this site. Based on available information and GA drawing, the maximum proposed cut for the connection road will be approximately 5.8 m deep (varying from 2.8 m to 5.8 m of earth cut from south to north). Within the zone of the proposed CN bridge, the base of cut at approximate Elevation 318.5. It is anticipated that the soils at the base of the cut will consist of compact to dense silty sand/sandy silt/sand and very stiff to hard silty clay. The earth cut will be formed through loose to compact silty sand to sand fill, loose to dense silty sand/sandy silt/sand and very stiff to hard silty clay. Part of the earth cut will be through the existing railway embankment. The fill type for the embankment is unknown. Due to access constraints and restrictions imposed by Metrolinx, no boreholes were advanced through the existing railway embankment. For this reason, the material that would be encountered while excavating through the existing embankment is unknown and boreholes should be advanced through the railway embankment prior to conducting design of the temporary protection/support systems by the party responsible for this work to obtain sufficient subsurface information. Boreholes should not only extend through the railway embankment. They should be drilled deep enough to confirm footing base elevation and design pile tip elevation. The groundwater level measured in the piezometers was 1.9 m below the ground surface (Elevation 320.3).

Although not investigated, railway embankment fill typically contains obstructions such as cobbles and boulders.

Based on the provided GA drawing the permanent cut slopes will be supported by the abutment walls and retaining walls.

Drainage will be required in the depressed section of the cut to remove water originating from:

- Storm runoff
- Seepage from the sides of the cut
- Cut below ground water level

Temporary drainage of the cuts should be provided to maintain a relatively dry, stable excavation. Positive drainage of the permanent cuts and road base must be provided.

The soils encountered at this site above the silty clay deposit (i.e. mostly above Elev. 315) are considered to be generally permeable and consequently seepage from the soil into the cut is expected to occur. It is recommended that this seepage be drained by means of the drains incorporated behind the abutments and by subdrains installed along each side of the connection road. The subdrains along the proposed road must be placed 1.4 m below the finished grade and must be lead to a positive outlet.

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. It is important to note that slopes steeper than 2H:1V may be subject to surficial instability which may include sloughing and gullying. Surface runoff and precipitation must be prevented from flowing perpendicularly down any slope surface. Erosion protection measures will have to be taken as necessary to maintain slope stability.

Further recommendations for cut and excavation are presented in Section 15.

If space is limited, temporary protection (shoring) will be required for the temporary earth cut operations. Recommendations for temporary protection (shoring) are presented in Section 18 of this report.

14. EAST/WEST RAILWAY APPROACH EMBANKMENTS

Within the area of the proposed CN bridge structure, the road connection grade will be near Elevation 318.5 and Metrolinx tracks will be at Elevation 326.0. Currently, at the site, the twin tracks are built in an embankment which is approximately 3.0 to 3.5 m high. It is not anticipated that the new fill will be placed to change the slope of the existing railway embankment.

All embankment fill must be constructed with adequate quality control in accordance with OPSS.PROV 206, OPSS.PROV 501, and AREMA Section 27.6.1 requirements and not contain medium or high plastic clay.

14.1 Slope Stability of Side Slope

The side slopes of the existing Railway embankments are not expected to be changed during the construction of the proposed Metrolinx Wellington Street to Victoria Street Connection proposed Metrolinx Wellington Street to Victoria Street Connection. If the existing slope is cut into or the slope angle is changed during construction a global slope stability analysis will need to be completed.

The global, internal and surficial stability of the approach embankment fills will depend on the slope geometry and also to a large degree on the material used to construct the embankments. Embankments constructed using granular material, select subgrade material or clean earth fill will have stable side slopes at inclinations of up to 2H:1V.

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14.2 Settlement

No settlement is expected since the approach embankments consist of an earth cut. If new fill is required to be placed to change the slopes of the existing railway embankment a settlement analysis will need to be completed.

15. TEMPORARY EXCAVATION

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.

Excavation for foundation construction will be extended through the loose to dense sand fill and silty sand fill and native compact to dense silty sand/sandy silt/sand. For the purposes of the OHSA, the native soils above the water table are classified as Type 3 soil. The native very stiff to hard silty clay deposit is classified as Type 2 soil. Cohesionless soils below the water table and fills are classified as Type 4 soil. A layer of organics was contacted below the cohesionless fill in Borehole CN16-14, and this soil layer is classified as Type 4 soil.

Obstructions such as cobbles, boulders, ballast, railway ties and/or other debris may be encountered during excavation within the embankment fill. The information provided by the borehole investigation is limited and therefore the potential presence of obstructions must be anticipated. Procedures to penetrate or remove these potential obstructions must be developed prior to the start of construction.

Development of the construction/excavation methodology must be carried out in consultation with Metrolinx/CN. Selection of the appropriate construction technique must take into account the need to avoid settlement and loss of ground below the rail tracks. The embankment surface and the track level and alignment should be monitored before, throughout and after cut/excavation to identify any induced settlement. The Contractor must be prepared to restore the track base and alignment if movement is detected.

The selection of the method of excavation is the responsibility of the contractor and must be based on his equipment, experience and interpretation of the site conditions. 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.

It is understood that a new Metrolinx bridge will be constructed approximately 35 m west of this site. All excavations must be carried out in a manner that avoids destabilising the foundations of the new bridge and slopes.

16. BACKFILL TO ABUTMENTS

For backfilling immediately behind the new abutment wall, 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, Granular B Type I may be used.

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 3102.100.

17. GROUNDWATER AND SURFACE WATER CONTROL

Piezometric levels obtained at this site indicate that the groundwater level is 1.9 m below the ground surface (Elevation 320.3). 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. 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.

Based on the grain size distribution curves, the coefficients of permeability (k) of the native soils are as follows:

Soil	Permeability, k (cm/sec)
Sand	1×10^{-2}
Sand and Silt till	$6.4\ 10^{-5}$
Silty clay and silty clay till	1×10^{-8}
Clayey silt till	1×10^{-7}

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), and OPSS. PROV 902 and NSSP FOUN0003. It is recommended that a pre-construction condition survey of existing structures within 100 m of the site be carried out prior to commencement of construction. It is recommended that a Professional Engineer with greater than 5 years of experience in designing dewatering systems be retained by the Contractor. The dewatering plan must be signed/sealed by the P.Eng.

The design of the dewatering system that may be required 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 or compacting engineered fill must be done in the dry. Dewatering must remain operational and effective until the footings are constructed and backfilled. Suggested wording for an NSSP in the regard is included in Appendix H.

18. RAIL TRACK PROTECTION AND SHORING

18.1 Rail Track Protection

Where open cut excavation is carried out, track protection should be supplied and designed in accordance with AREMA Section 28.1.5. Discussions with the railway authorities should be carried out to determine the required performance level of protection. Metrolinx Rail may require a more stringent performance level for railway protection.

It is anticipated that full closure of the twin rail tracks might not be an alternative for construction of the new bridge. Therefore, consideration should be given to develop and implement a staged construction plan at this site, which allows to maintain at least one of the rail tracks operating during construction of the new bridge. The design of railway protection should be the responsibility of the Contractor. However, potential options for use as temporary shoring/railway protection at this site include the installation of a caisson wall, soldier pile and lagging or sheet pile wall to support the rail tracks during construction. Potential obstructions in the existing embankment fill may result in difficulty driving sheet piles. The type and construction method of the rail track protection selected must consider constructability aspects, the impact on the railway tracks, and the risks associated with track movement during excavation under an operating railway. This would be achieved through the following construction sequence:

1. Install the rail track protection system below the existing twin rail tracks to support them during excavation of permanent cut and/or excavation and bridge construction.
2. Close one of the twin tracks, and maintain one of them operating.
3. Construct half portion of the new bridge in the zone were the tracks are closed.
4. Once this half of the bridge is completed, proceed to switch to the other rail track (open the rail tracks that were closed, and close the rail tracks that were open).
5. Built the second half of the bridge.

If closing of one track at a time is not an option at this site, then tunnelling should be considered such as a jack/push box tunnel.

It is recommended that the rail track protection will be planned in conjunction with the other proposed Metrolinx bridge to be constructed approximately 30 t0 40 m west of this site. This westerly bridge will accommodate the proposed S-E Ramp to pass under the existing twin Metrolinx tracks.

The number of construction stages should be kept to a minimum in order to reduce the bridges cost, construction duration and any disruption to the rail operations.

All rail track protection should be designed by a Professional Engineer experienced in such designs.

18.2 Preliminary Geotechnical Parameters for Temporary Shoring

Shoring is required at this site during earth cut operations. Where open cut excavation is carried out, track protection should be supplied in accordance with AREMA Section 28.1.5. The required performance level should be confirmed with Metrolinx. Such performance level must be confirmed with Metrolinx. The temporary shoring design must also meet the requirements in AREMA Section 28.1.5.

The design of track protection should be the responsibility of the Contractor. The material supported by the structure walls will consist of the existing embankment fills. Due to drilling constrains within the rail corridor, soil information was not able to be obtained for the existing embankment fill. It is recommended that additional boreholes through the embankments be advanced by the party responsible for the design of the temporary protection/support systems to obtain sufficient subsurface data prior to the design and construction. Preliminary lateral earth pressures may be calculated using the parameters given below, however, it must be noted that boreholes will need to be drilled to confirm the consistency and strength of the railway embankment fill. The below given values are for flat ground behind the shoring. If there is any sloping fill behind the shoring the lateral earth pressures must be revisited.

γ	=	21 kN/m^3(Fills above GWL)
	=	20 kN/m^3(Native cohesionless soils above GWL)
γ_w	=	11 kN/m^3 (Fills below GWL)
	=	10 kN/m^3 (Native cohesionless soils below GWL)
	=	9 kN/m^3 (Native cohesive soils below GWL)
K_a	=	0.35 (Embankment fills)
	=	0.33 (Loose to compact silty sand to sand fill)
	=	0.31 (Compact native sand, silty sand, sandy silt)
	=	0.33 (Very stiff to hard silty clay)
K_o	=	0.52 (Embankment fills)
	=	0.50 (Loose to compact silty sand to sand fill)
	=	0.47 (Compact native sand, silty sand, sandy silt)
	=	0.50 (Very stiff to hard silty clay)
K_p	=	2.9 (Embankment fills)
	=	3.0 (Loose to compact silty sand to sand fill)
	=	3.3 (Compact native sand, silty sand, sandy silt)
	=	3.0 (Very stiff to hard silty clay)

The design water level of Elevation 321.0 m is recommended.

The actual pressure distribution acting on the shoring system is a function of the construction sequence, and the relative flexibility of the wall and these factors must be considered when designing the shoring system. The design of all members of the shoring system should include the effects of surcharge loads such as those imposed by construction equipment and railway traffic. All shoring systems must be designed by a Professional Engineer experienced in such designs.

19. 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. The stratigraphy of the site consists of embankment fill over loose to dense silty sand to sand fill, overlying native compact to dense silty sand, sandy silt and upper sand, and compact gravelly sand. Below the cohesionless soils, an extensive deposit of very stiff to hard silty clay was contacted, underlain by very dense sandy silt 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 6.14.7.2 of the CHBDC 2019, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 19.1 may be used:

Table 19.1 – 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 review of the SPT data, seismically induced liquefaction of foundation soils is not considered to be a concern at this site.

20. ADJACENT STRUCTURES, RAIL TRACKS AND BURIED UTILITIES

The potential presence of underground utilities at the site must be confirmed prior to construction. It is recommended that the exact locations and elevations of any utilities be established by the designer, and compared with the extent of the potential work zones related to the foundations of the proposed replacement structures and associated works. Protection and/or relocation of utilities may be required. Underground utilities should not be undermined or damaged during new foundation construction.

Monitoring of the railway as well as any nearby underground utility and structures must be carried out during construction to identify any areas of settlement. A settlement monitoring program for construction under the Metrolinx right-of-way will need to be designed and implemented in accordance with Metrolinx's requirements. This program must be developed prior to construction for Metrolinx's review and approval. The monitoring of track settlement should be accomplished by means of surface and subsurface settlement points.

If pile driving is required close to adjacent structure(s), the following recommendations should be carried out prior to commencement of foundation construction:

- Implement a vibration and settlement monitoring program during and after construction of the new abutments to assess any potential adverse impact on the existing operating structure or railway tracks.
- Inspection of the existing operating structure during foundation construction to monitor if there is any movement or distress.
- The structural designers should assess the magnitude of settlement or horizontal displacement that would constitute a concern for the stability or serviceability of the existing operational structures. These limits should be incorporated into the monitoring program as review and alert levels.
- Carry out post-construction condition survey.

21. CORROSION AND SULPHATE ATTACK POTENTIAL

The results of the corrosivity and sulphate analytical tests conducted on the existing silty sand fill, indicates the following conditions at the location tested:

- The potential for sulphate attack on concrete foundations from the surrounding soils is considered to be negligible due to the low concentration of sulphate and chloride in the samples tested. The selection of class of concrete should consider the effects of the road de-icing salts.

- The potential for soil corrosion on metal is considered to be moderate.
- 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.

22. CONSTRUCTION CONCERNS

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

1. Protection of the Existing Rail Tracks

   It is anticipated that during the staged construction of the new bridge, one of the twin tracks will remain in service. The Contractor must provide adequate protection/support to ensure that the performance of the rail tracks are not compromised and are protected.

2. Pile Installation

   Occasional cobbles and boulders were encountered in the boreholes during drilling operations (e.g. tri-cone grinding). Glacial till deposits inherently contain cobbles and boulders. Hard driving conditions through the very dense soils should be expected. Pile tips should be reinforced with Titus steel (Standard H-point) to protect the driven piles from damage.

3. Excavation

   Hydraulic equipment is expected to be capable of excavating to the required depths at this site. If excavations advance below the existing groundwater level, groundwater control measures will have to be implemented in order to maintain stables sides and base in the excavation.

   The glacial till contain cobbles and boulders. Equipment selected for excavation must be capable of penetrating, handling and/or removing these obstructions.

   No boreholes were drilled through the railway embankment and therefore it is unknown what material the embankment is comprised of. Obstructions such as cobbles, boulders, ballast, railway ties and/or other debris may be encountered during excavation within the railway embankment fill. Boreholes are recommended to be drilled through the railway embankment by the party responsible for the design of the temporary protection/support systems before the design is carried out.

4. Impact of excavation on the rail tracks and embankment

   Daily visual inspection and settlement monitoring of the rail tracks and rail track embankment must be carried out in the vicinity of the construction works. If any soil loss, track damage or settlement is observed to occur, these matters must immediately be brought to the attention of the Metrolinx CA for determining if further action is required. The Contractor must be prepared to work with Metrolinx to restore the track base and alignment if movement is detected.

5. Groundwater Control and Impacts

   Seepage and perched groundwater will be encountered within the cohesionless fill and native sand/silty sand/sandy silt above the cohesive deposit. The impact of seepage or surface water could destabilize the sides and or base of the excavation. The Contractor's unwatering plan must be available for rapid implementation should the need arise. Proper groundwater and surface water control measures must be in place prior to commencing excavation. All footings/pile caps must be constructed in the dry. Groundwater control measures such as perimeter ditches and pumping from filtered sumps for nominal penetration below the groundwater level, sheeted excavation (cofferdam) or vacuum well-points should be implemented to remove any accumulation of water from the pile cap base/or footings prior to placing concrete. Surface runoff and precipitation should be diverted away from the excavations at all times. The Contractor's unwatering plan must be in place prior to commencing excavation. All footings/pile cap must be constructed in the dry.

   The potential impact of drainage of the permanent cuts on the local groundwater table must be addressed by a hydrogeologist, who should also consider the need to apply for an MOE Permit to Take Water (PTTW).

6. Environmental Investigation

   Soil samples obtained within the cohesionless fill and native cohesionless soils revealed strong gasoline odour. It is recommended that environmental/analytical screening and testing be conducted at this site to determine the quality of the excess excavated soils for soil management purposes (re-use on site and/or off-site disposal). Environmental testing of groundwater should also be conducted for the purpose of PTTW application.

7. Existing Slopes and Cut Slopes

The railway embankment side slopes should be inspected before and after construction and any surficial disturbance should be documented. Where necessary, remedial measures such as re-vegetation and/or placement of gravel sheeting may be required.

For temporary earth cut, the slopes should be inspected for surficial disturbance.

23. CLOSURE

Engineering analysis and preparation of the report were carried out by Dr. Nancy Berg and Ms. R. Palomeque Reyna, 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.

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Appendix A

Record of Borehole Sheets and Laboratory Test Results
Present Investigation

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

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

$\underline{\blacktriangledown}$ 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.

UNIFIED SOILS CLASSIFICATION

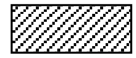
MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, 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			

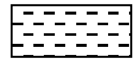
EXPLANATION OF ROCK LOGGING TERMS

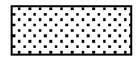
ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

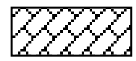
SYMBOLS


CLAYSTONE


SILTSTONE


SANDSTONE


COAL


Bedrock (general)

DISCONTINUITY SPACING

Bedding	**Bedding Plane Spacing**
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength**		**Field Estimation of Hardness***
	(MPa)	**(psi)**	
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No CN16-13

METRIC

GWP# 408-88-00 | LOCATION MTM NAD 83 Zone 10: N 4 814 207.8 E 226 304.9 | ORIGINATED BY BL

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

DATUM Geodetic | DATE 2019.07.09 - 2019.07.09 | LATITUDE 43.463650 | LONGITUDE -80.470080 | CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
322.9	GROUND SURFACE					
0.0	**TOPSOIL:** (150mm)					
0.2	Silty **SAND**, gravelly, some clay, occasional cobbles, occasional decays wood fragments Compact to Loose Brown Moist (FILL)	1	SS	15		
		2	SS	10	322	22 37 30 11
		3	SS	5	321	
		4	SS	18		Switch to tricone
319.9 3.0	Silty **SAND**, trace to some gravel Dense Brown Moist	5	SS	41	320	
318.8 4.1	Silty **CLAY**, trace sand Very Stiff to Hard Brown Moist				319	
	Grey	6	SS	26	318	
		7	SS	35	317	0 7 39 54
		8	SS	31	316, 315	
		9	SS	25	314, 313	

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity; 20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No CN16-13

METRIC

GWP# 408-88-00 LOCATION MTM NAD 83 Zone 10: N 4 814 207.8 E 226 304.9 ORIGINATED BY BL

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

DATUM Geodetic DATE 2019.07.09 - 2019.07.09 LATITUDE 43.463650 LONGITUDE -80.470080 CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page						
	Silty **CLAY**, trace sand Very Stiff Grey Moist						
		10	SS	22	312	~20	
311.2 / 11.7	Sandy **SILT**, trace clay Very Dense Grey Moist (TILL)				311		
		11	SS	76		~12	0 31 61 8
	Moist to Wet				310		
		12	SS	100/0.275	309	~22	
308.1 / 14.8	Silty **CLAY**, trace sand Hard Grey Moist				308		
		13	SS	84		~20	
307.1 / 15.8	END OF BOREHOLE AT 15.8m. BOREHOLE CAVED IN AND IT WAS NOT POSSIBLE TO MEASURE THE WATER LEVEL UPON COMPLETION. BOREHOLE BACKFILLED WITH CEMENT AND GROUT TO 0.6m, THEN BENTONITE HOLEPLUG TO SURFACE.						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

+ ³, × ³ : Numbers refer to Sensitivity   15-20-5-10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No CN16-14

METRIC

GWP# 408-88-00 | LOCATION MTM NAD 83 Zone 10: N 4 814 200.2 E 226 312.3 | ORIGINATED BY BL

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

DATUM Geodetic | DATE 2019.07.16 - 2019.07.17 | LATITUDE 43.463566 | LONGITUDE -80.469959 | CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
323.2	GROUND SURFACE						
0.0 / 0.1	**TOPSOIL:** (100mm)						
	Silty **SAND**, trace gravel, trace clay, Compact to Loose, Brown, Moist (FILL)	1	SS	18		323	
		2	SS	8		322	
		3	SS	5			4 75 17 4
		4	SS	23		321	
320.2 / 3.0	**ORGANICS**, occasional roots and rootlets, Compact, Black, Wet	5	SS	15		320	Switch to tricone
319.7 / 3.5	Silty **SAND**, trace to some gravel, Compact to Dense, Brown, Moist to Wet				∇	319	
		6	SS	26		318	
		7	SS	32		317	
316.0 / 7.2	Silty **CLAY**, trace sand, Hard, Brown, Moist					316	
		8	SS	36		315	
	Grey	9	SS	37		314	

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 (124)

UNIT WEIGHT γ kN/m³

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No CN16-14 METRIC

GWP# 408-88-00 LOCATION MTM NAD 83 Zone 10: N 4 814 200.2 E 226 312.3 ORIGINATED BY BL

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

DATUM Geodetic DATE 2019.07.16 - 2019.07.17 LATITUDE 43.463566 LONGITUDE -80.469959 CHECKED BY RPR

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, trace sand Hard to Very Stiff Grey Moist				313		
		10	SS	38			0 2 32 66
					312		
					311		
		11	SS	23			
					310		
		12	SS	83			
					309		
308.5 14.7	**SAND** and **SILT**, trace clay Very Dense Grey Wet (TILL)						
					308		
		13	SS	100/ 0.250			0 44 49 7
					307		
306.7 16.5	Silty **CLAY**, trace sand Hard Grey Moist						
		14	SS	32			
					306		
					305		
		15	SS	79			
					304		

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

Continued Next Page

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

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

RECORD OF BOREHOLE No CN16-14

METRIC

GWP# 408-88-00	LOCATION MTM NAD 83 Zone 10: N 4 814 200.2 E 226 312.3		ORIGINATED BY BL
DIST HWY 7	BOREHOLE TYPE Hollow Stem Augers/Tricone		COMPILED BY BH
DATUM Geodetic	DATE 2019.07.16 - 2019.07.17	LATITUDE 43.463566 LONGITUDE -80.469959	CHECKED BY RPR

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/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
	Silty **CLAY**, trace sand Hard Grey Moist		16	SS	59		303				
							302				
							301				
			17	SS	75		300				
							299				
							298				
			18	SS	41		297				
							296				
							295				
			19	SS	48		294	0	1	45	54

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No CN16-14

METRIC

GWP# 408-88-00 | LOCATION MTM NAD 83 Zone 10: N 4 814 200.2 E 226 312.3 | ORIGINATED BY BL

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

DATUM Geodetic | DATE 2019.07.16 - 2019.07.17 | LATITUDE 43.463566 | LONGITUDE -80.469959 | CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page						
	Silty **CLAY**, trace sand Hard Grey Moist				293		
292.4 30.8	Sandy **SILT**, trace gravel, trace clay, occasional cobbles Very Dense Grey Moist (TILL)				292		
		20	SS	100/ 0.100	291	o	Tricone grinding
					290		
		21	SS	100/ 0.125	289	o	
288.0 35.2	END OF BOREHOLE AT 35.2m. BOREHOLE OPEN AND WATER LEVEL AT 4.6m UPON COMPLETION. BOREHOLE BACKFILLED WITH CEMENT, GRAVEL AND BENTONITE TO SURFACE.	22	SS	100/ 0.125		o	

STRAT PLOT | GROUND WATER CONDITIONS | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa: O 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

+ 3, × 3: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No CN16-15

METRIC

GWP# 408-88-00	LOCATION MTM NAD 83 Zone 10: N 4 814 168.4 E 226 309.6		ORIGINATED BY BL
DIST HWY 7	BOREHOLE TYPE Hollow Stem Augers/Tricone		COMPILED BY BH
DATUM Geodetic	DATE 2019.07.18 - 2019.07.19	LATITUDE 43.463365 LONGITUDE -80.470109	CHECKED BY RPR

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
322.1	GROUND SURFACE
0.0	**ORGANICS:** (100mm)
0.1	Silty **SAND**, mixed with organics, some gravel, trace clay Loose to Very Loose Black to Brown Moist (FILL) Compact Wet
319.1 / 3.0	**SAND**, trace silt, trace clay Dense to Compact Brown Wet
314.9 / 7.2	Clayey **SILT**, with sand, trace gravel Very Stiff to Hard Grey Moist to Wet (TILL)

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	9	11 55 28 6
2	SS	2	
3	SS	1	15 48 32 5
4	SS	11	Switch to tricone
5	SS	25	Tricone grinding
6	SS	33	0 91 8 1
7	SS	20	
8	SS	29	Tricone grinding
9	SS	45	4 25 46 25

Elevation scale: 322, 321, 320, 319, 318, 317, 316, 315, 314, 313

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 (sample 4: 98)
UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity   20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No CN16-15

METRIC

GWP# 408-88-00 | LOCATION MTM NAD 83 Zone 10: N 4 814 168.4 E 226 309.6 | ORIGINATED BY BL

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

DATUM Geodetic | DATE 2019.07.18 - 2019.07.19 | LATITUDE 43.463365 | LONGITUDE -80.470109 | CHECKED BY RPR

SOIL PROFILE — Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	Clayey **SILT**, with sand, trace gravel Hard Grey Moist to Wet (TILL)				312		
		10	SS	43	311	~20	No recovery
310.6 11.5	Silty **CLAY**, trace sand Hard Grey Moist to Wet				310		
		11	SS	39		~32	
					309		
		12	SS	37	308	~20	
					307		
		13	SS	52		~26	
					306		
		14	SS	61	305	~20	
					304		
		15	SS	71		~20	
					303		

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 γ kN/m³. GR SA SI CL. GROUND WATER CONDITIONS. STRAT PLOT.

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No CN16-15

METRIC

GWP# 408-88-00 LOCATION MTM NAD 83 Zone 10: N 4 814 168.4 E 226 309.6 ORIGINATED BY BL

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

DATUM Geodetic DATE 2019.07.18 - 2019.07.19 LATITUDE 43.463365 LONGITUDE -80.470109 CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page							
	Silty **CLAY**, trace sand Hard Grey Moist to Wet		16	SS	100/ 0.300		302	
							301	
							300	
			17	SS	53		299	0 2 23 75
							298	
							297	
			18	SS	37		296	
							295	
294.7 27.4	**SAND**, some gravel, some silt, trace clay Very Dense Brown Moist						294	
			19	SS	80		293	

Continued Next Page

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

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

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

Ministry of Transportation Ontario

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RECORD OF BOREHOLE No CN16-15

METRIC

GWP# 408-88-00 | LOCATION MTM NAD 83 Zone 10: N 4 814 168.4 E 226 309.6 | ORIGINATED BY BL

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

DATUM Geodetic | DATE 2019.07.18 - 2019.07.19 | LATITUDE 43.463365 | LONGITUDE -80.470109 | CHECKED BY RPR

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	SAND, some gravel, some silt, trace clay Very Dense Brown Moist				292		Tricone grinding
291.0 / 31.1	Sandy **SILT**, trace clay, trace gravel Very Dense Grey Moist (TILL)				291		
		20	SS	100/0.100	290	~13	Tricone grinding
					289		
	Occasional cobbles	21	SS	100/ 0.075		~6	Tricone grinding
287.7 / 34.4	**SAND** and **GRAVEL** Very Dense Grey Moist				288		
286.9 / 35.2		22	SS	100/ 0.100	287	~6	
	END OF BOREHOLE AT 35.2m. BOREHOLE CAVED IN AND IT WAS NOT POSSIBLE TO MEASURE THE WATER LEVEL UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG, SAND, AND CEMENT TO SURFACE.						

Legend: 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; GR SA SI CL

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

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

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RECORD OF BOREHOLE No CN16-16 — 1 OF 2 — METRIC

GWP# 408-88-00 | LOCATION MTM NAD 83 Zone 10: N 4 814 160.0 E 226 308.9 | ORIGINATED BY BL
DIST ____ HWY 7 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY BH
DATUM Geodetic | DATE 2019.07.02 - 2019.07.02 | LATITUDE 43.463263 | LONGITUDE -80.470014 | CHECKED BY RPR

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
322.2	GROUND SURFACE
0.0	**TOPSOIL:** (100mm)
0.1	**SAND**, some silt, some gravel, occasional organics; Dense; Brown; Moist; (FILL)
321.5 / 0.7	Silty **SAND**, mixed with organics, some gravel, trace clay, gasoline odour; Loose; Black; Moist; (FILL)
319.2 / 3.0	Silty **SAND**, trace gravel, trace clay; Compact; Brown; Moist
316.6 / 5.6	Gravelly **SAND**, trace silt, trace clay; Compact; Brown; Wet
315.0 / 7.2	Silty **CLAY**, trace sand; Very Stiff; Grey; Moist

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	34
2	SS	24
3	SS	8
4	SS	19
5	SS	17
6	SS	11
7	SS	15
8	SS	24
9	SS	26

REMARKS & GRAIN SIZE DISTRIBUTION (%) (Sample 2): GR 13, SA 64, SI 20, CL 3

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (322, 321, 320, 319, 318, 317, 316, 315, 314, 313); 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

Continued Next Page

+ ^{3}, × ^{3}: Numbers refer to Sensitivity — 20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-11375(GINTDATA).GPJ 2017TEMPLATE(MTO).GDT 2/18/20

RECORD OF BOREHOLE No CN16-16

METRIC

GWP# 408-88-00	LOCATION MTM NAD 83 Zone 10: N 4 814 160.0 E 226 308.9		ORIGINATED BY BL
DIST HWY 7	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY BH
DATUM Geodetic	DATE 2019.07.02 - 2019.07.02	LATITUDE 43.463263 LONGITUDE -80.470014	CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION (Continued From Previous Page)	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	Silty **CLAY**, trace sand Very Stiff to Hard Grey Moist				312					
		10	SS	18	311		0	7	34	59
		11	SS	20	310					
		12	SS	35	309, 308					
306.3		13	SS	33	307					
15.8	END OF BOREHOLE AT 15.8m. WATER LEVEL AT 4.0m UPON COMPLETION. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 3.04m slotted screen.									

WATER LEVEL READINGS

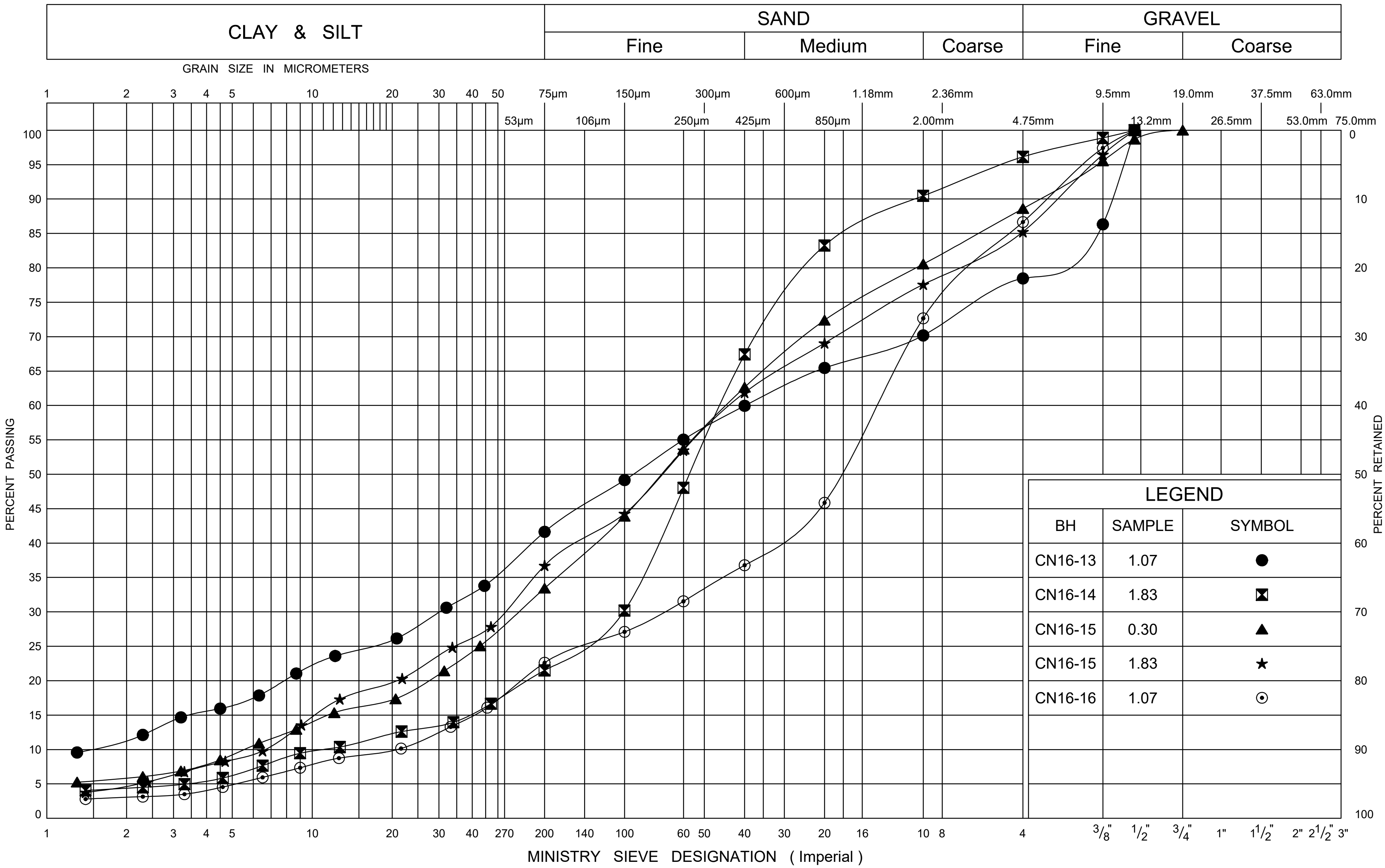
DATE	DEPTH(m)	ELEV.(m)
2019.08.08	1.9	320.3
2019.08.29	1.9	320.3

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

+³, ×³: Numbers refer to Sensitivity

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

78 12 M



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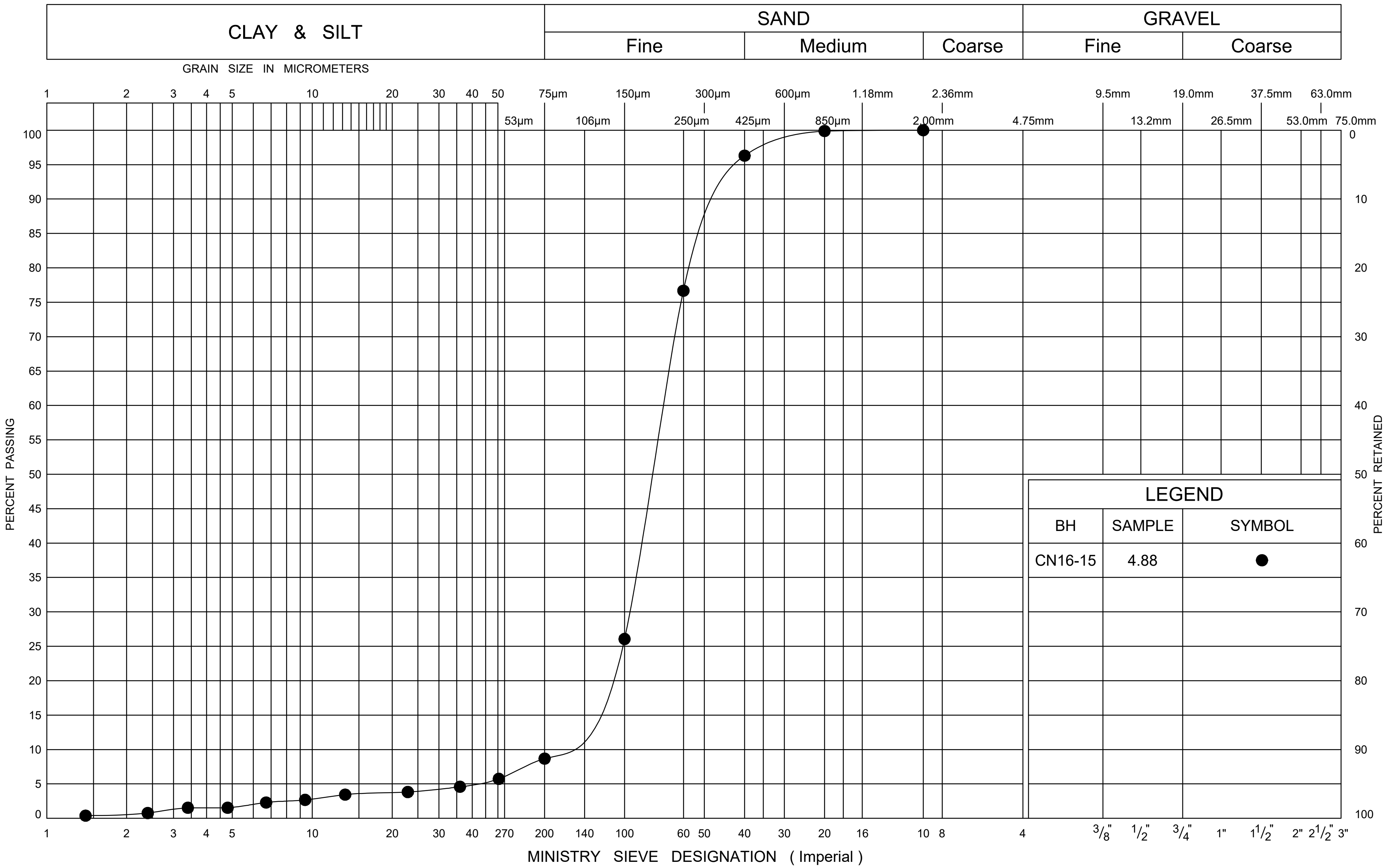
GRAIN SIZE DISTRIBUTION

Silty SAND FILL

FIG No A1

W P 408-88-00



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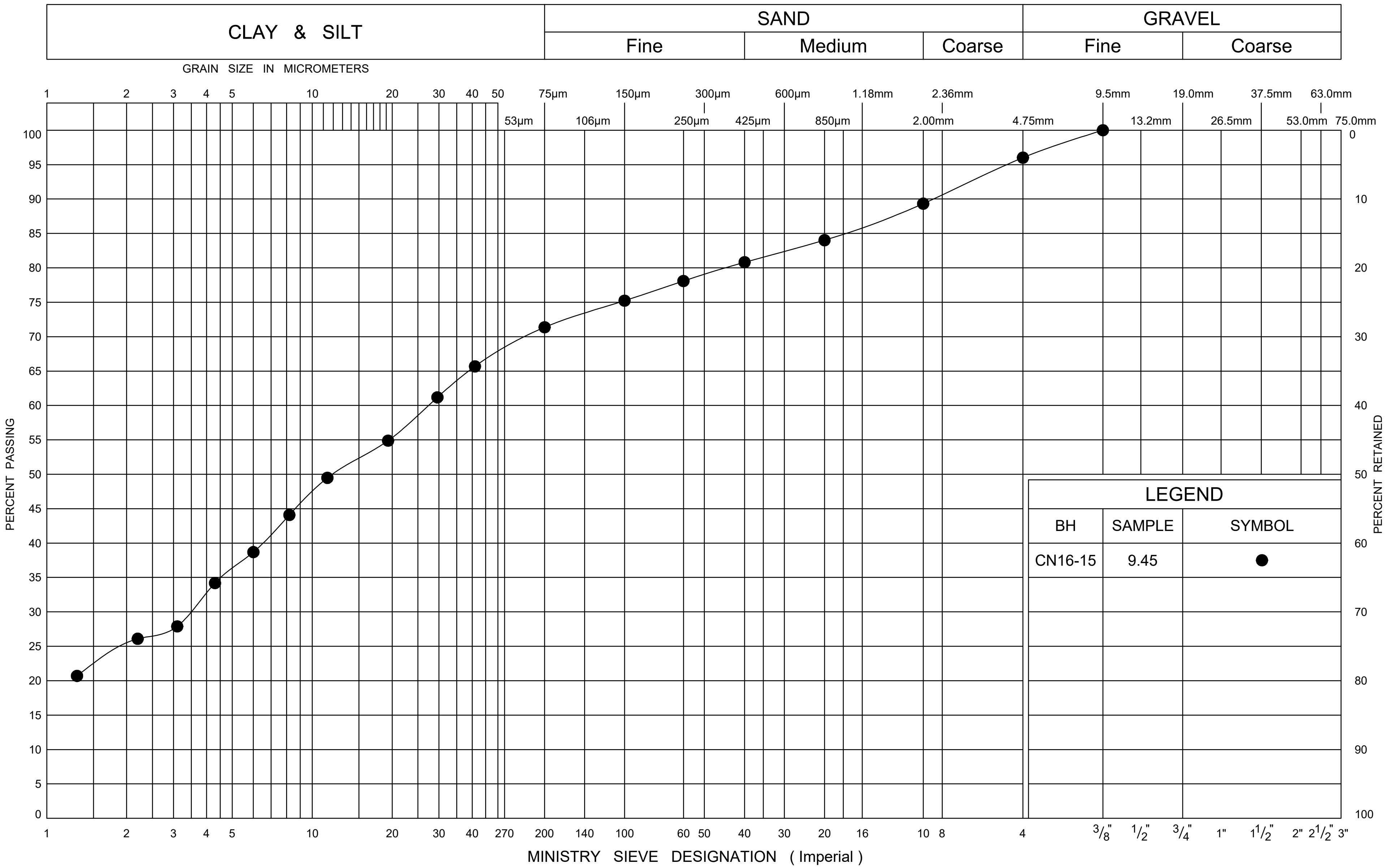
GRAIN SIZE DISTRIBUTION

SAND

FIG No A2

W P 408-88-00

78 12 M



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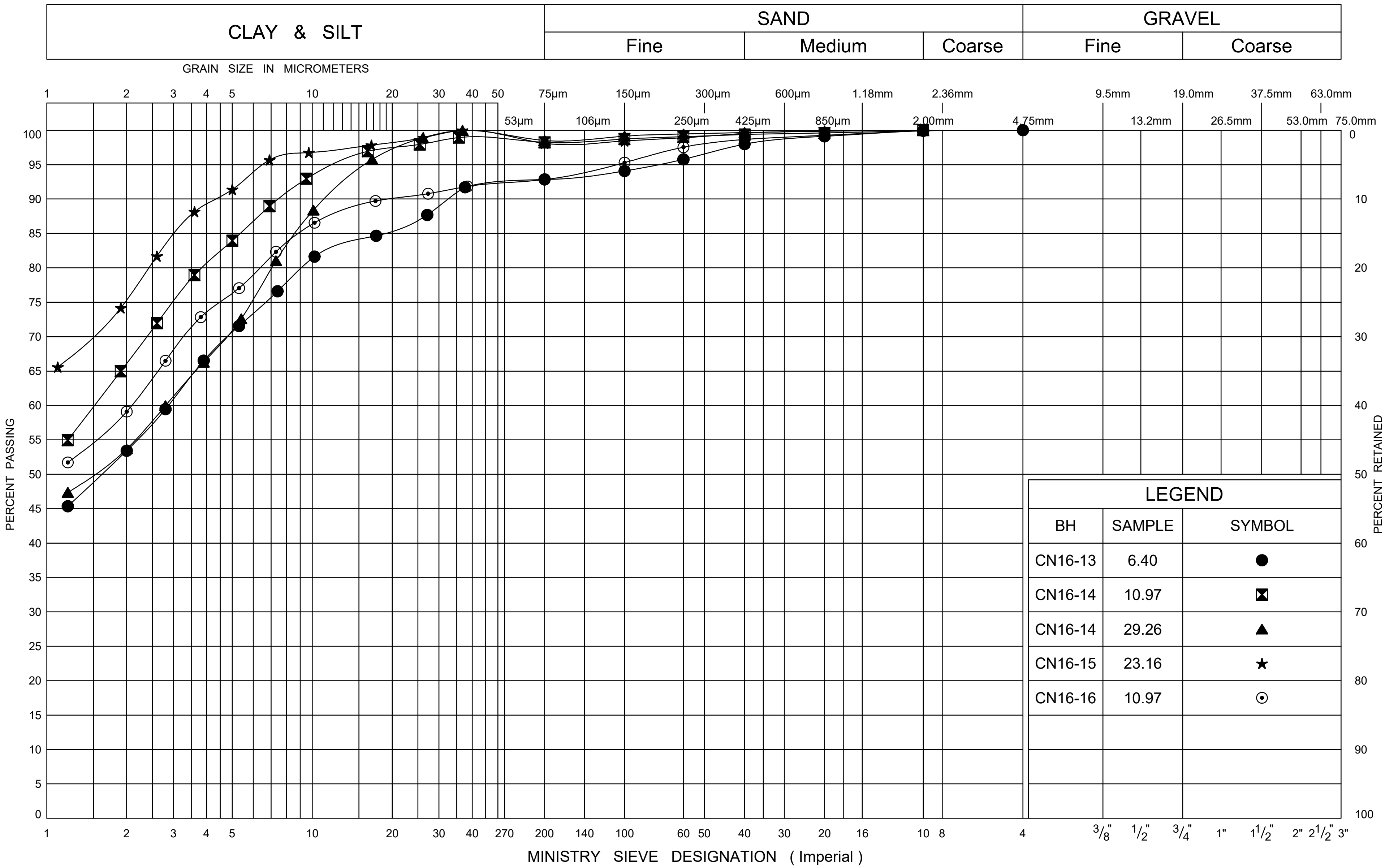
GRAIN SIZE DISTRIBUTION

Clayey SILT TILL

FIG No A3

W P 408-88-00

78 12 M



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GRAIN SIZE DISTRIBUTION

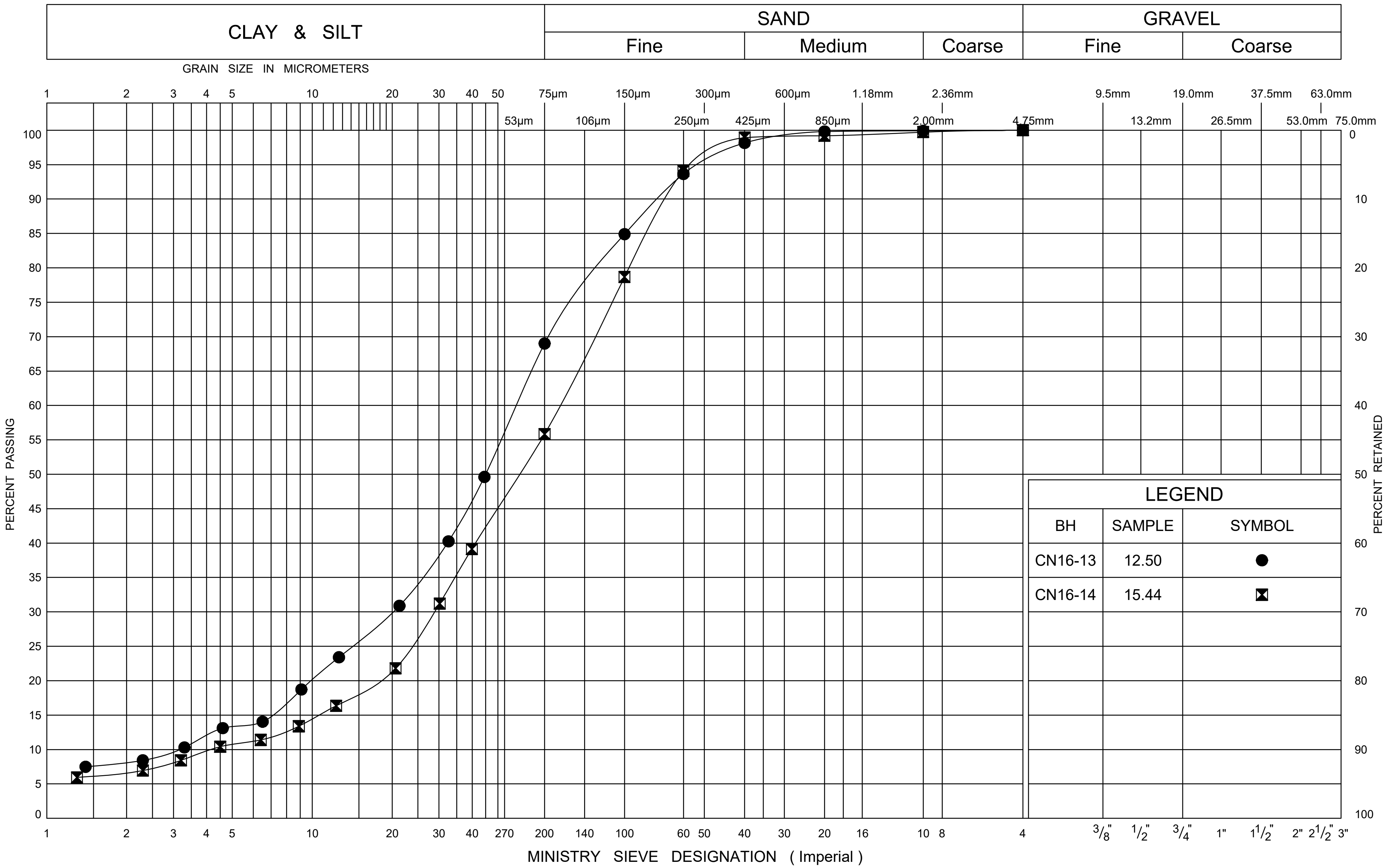
Silty CLAY

FIG No A4

W P 408-88-00

ONTARIO MOT GRAIN SIZE 2 MTO-11375(GINTDATA).GPJ ONTARIO MOT.GDT 1/7/20

78 12 M



BH	SAMPLE	SYMBOL
CN16-13	12.50	●
CN16-14	15.44	⊠

ONTARIO MOT GRAIN SIZE 2 MTO-11375(GINTDATA).GPJ ONTARIO MOT.GDT 1/7/20

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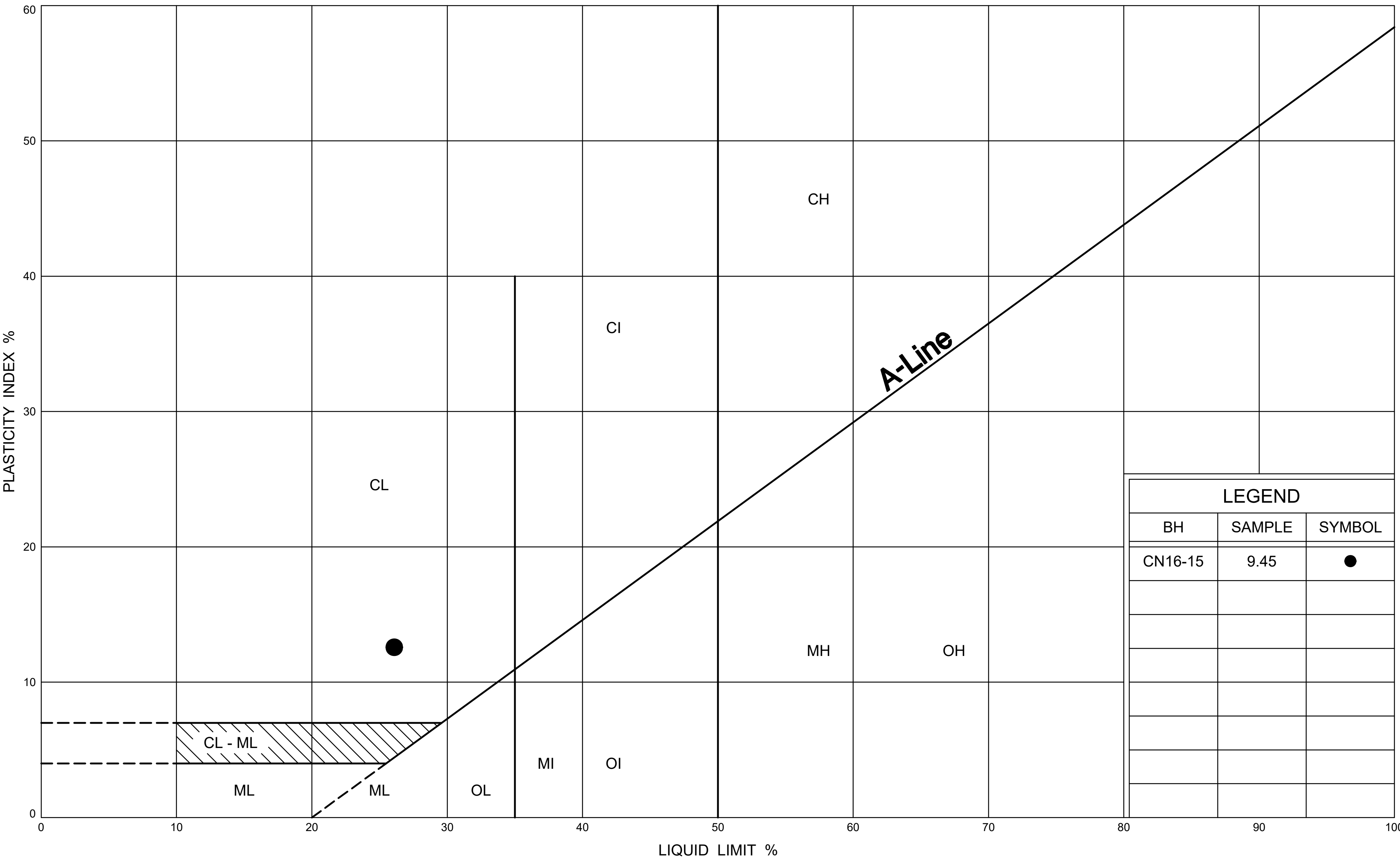
GRAIN SIZE DISTRIBUTION
SAND and SILT TILL / Sandy SILT TILL

FIG No A5

W P 408-88-00



LEGEND		
BH	SAMPLE	SYMBOL
CN16-15	9.45	●

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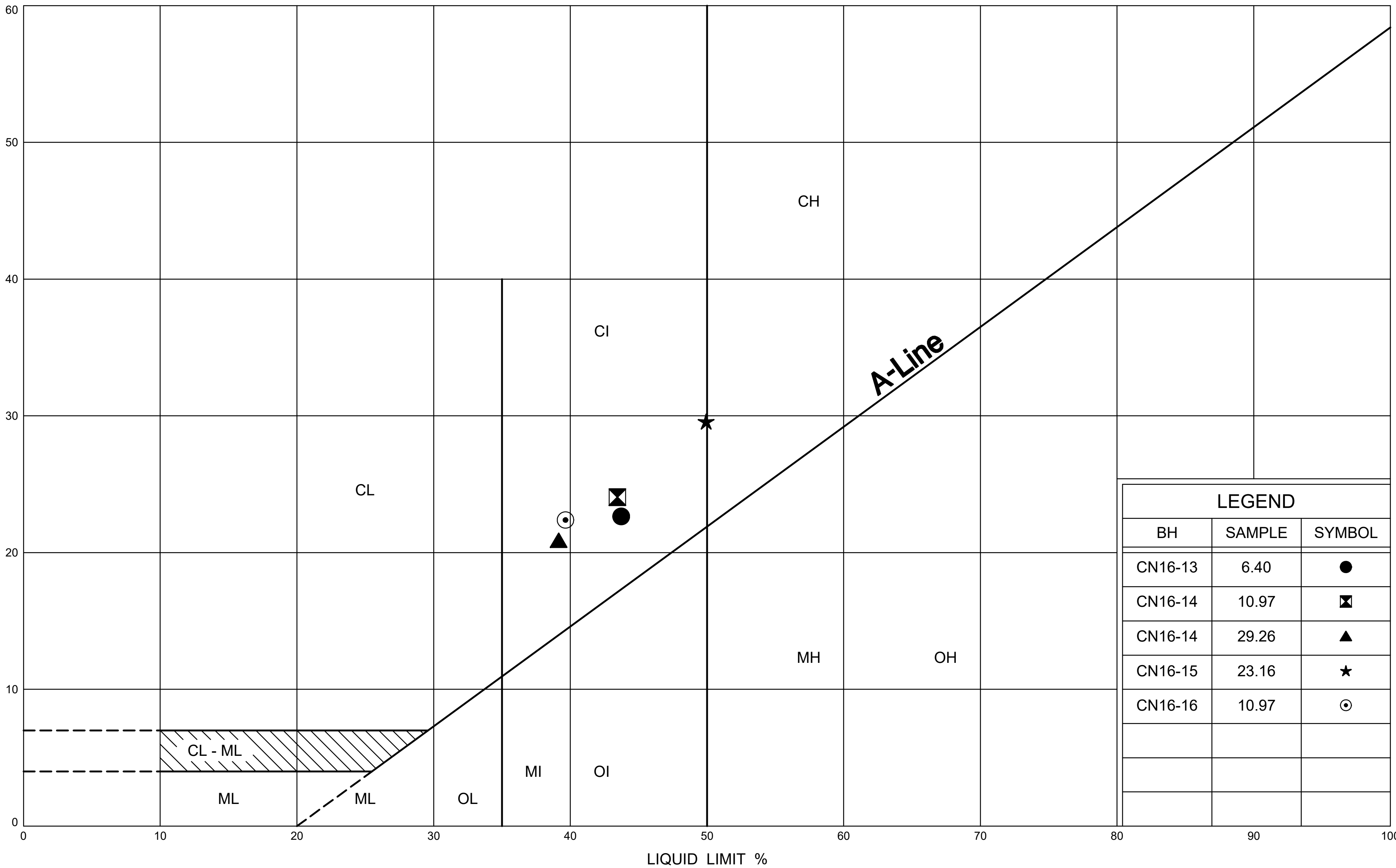
PLASTICITY CHART

Clayey SILT TILL

FIG No A6

W P 408-88-00

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
CN16-13	6.40	●
CN16-14	10.97	⧗
CN16-14	29.26	▲
CN16-15	23.16	★
CN16-16	10.97	⊙

ONTARIO MOT PLASTICITY CHART MTO-11375(GINTDATA).GPJ ONTARIO MOT.GDT 1/7/20

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PLASTICITY CHART
Silty CLAY

FIG No A7

W P 408-88-00

THURBER

Appendix B

Record of Borehole Sheets and Laboratory Test Results
Previous investigation

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.

UNIFIED SOILS CLASSIFICATION

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			

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

Bedrock (general)

DISCONTINUITY SPACING

Bedding	**Bedding Plane Spacing**
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength** (MPa)	(psi)	**Field Estimation of Hardness***
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

THURBER

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 08-046

METRIC

G.W.P. 408-88-00
LOCATION N 4 814 170.54 E 226 315.49
ORIGINATED BY SA
HWY 7
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY FK
DATUM Geodetic
DATE 2008.08.07 - 2008.08.11
CHECKED BY RPR

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X 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

ELEV DEPTH	DESCRIPTION
321.9 / 0.0	**SAND**, trace gravel, some silt, occasional topsoil Dark Brown to Black Strong Gasoline Odour Compact Moist (FILL)
319.9 / 2.1	Layer of black sandy silt (100mm) Very Loose Black
	Sandy **SILT**, trace to some gravel, occasional cobbles, gasoline odour Loose to Compact Grey to Brown Wet
315.5 / 6.4	Clayey **SILT**, some sand, trace gravel, occasional silty sand seams Very Stiff Brown to Grey (TILL) Hard Grey
312.8 / 9.1	Silty **CLAY**, trace sand Hard Grey

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	13				
2	SS	3				
3	SS	7				
4	SS	16	2	44	50	4
5	SS	28				
6	SS	23				
7	SS	34	1	18	54	27
8	SS	39				

Elevation scale: 322, 321, 320, 319, 318, 317, 316, 315, 314, 313

Continued Next Page

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

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

RECORD OF BOREHOLE No 08-046

METRIC

G.W.P. 408-88-00 LOCATION N 4 814 170.54 E 226 315.49 ORIGINATED BY SA
HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY FK
DATUM Geodetic DATE 2008.08.07 - 2008.08.11 CHECKED BY RPR

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty CLAY, trace sand Very Stiff to Hard Grey				312	
		9	SS	24	311	
		10	SS	32	310	
					309	
		11	SS	42	308	0 3 40 58
					307	
		12	SS	70	306	
		13	SS	100/ .150	305	
					304	
		14	SS	90	303	

Continued Next Page

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

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 08-046

METRIC

G.W.P. 408-88-00
LOCATION N 4 814 170.54 E 226 315.49
ORIGINATED BY SA

HWY 7
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY FK

DATUM Geodetic
DATE 2008.08.07 - 2008.08.11
CHECKED BY RPR

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	Silty CLAY, trace sand, Hard, Grey	15	SS	89	302	~18 (wp 18 – wL 40)	0	1	34	65
		16	SS	52	299	~22				
		17	SS	35	296	~32				
	occasional silt seams	18	SS	50	294	~22				
293.3 / 28.7	SAND, some silt, trace clay, Very Dense, Grey, Wet	19	SS	79	293	~9	5	69	26 (SI+CL)	

SHEAR STRENGTH kPa: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 08-046

METRIC

G.W.P. 408-88-00	LOCATION N 4 814 170.54 E 226 315.49	ORIGINATED BY SA	
HWY 7	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY FK	
DATUM Geodetic	DATE 2008.08.07 - 2008.08.11	CHECKED BY RPR	

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	Continued From Previous Page										
291.8 / 30.2	Sandy SILT, some clay, trace gravel, occasional cobbles; Very Dense; Grey; Moist; (TILL)	20	SS	100/ .050	292 / 291	o		4	35	43	19
		21	SS	100/ .075	290 / 289	o					
288.2		22	SS	80/ .050		o					
33.7	END OF BOREHOLE AT 33.7m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. Piezometer destroyed										

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 O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X 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; REMARKS & GRAIN SIZE DISTRIBUTION (%)

ONTMT4S 6417R.GPJ 3/12/09

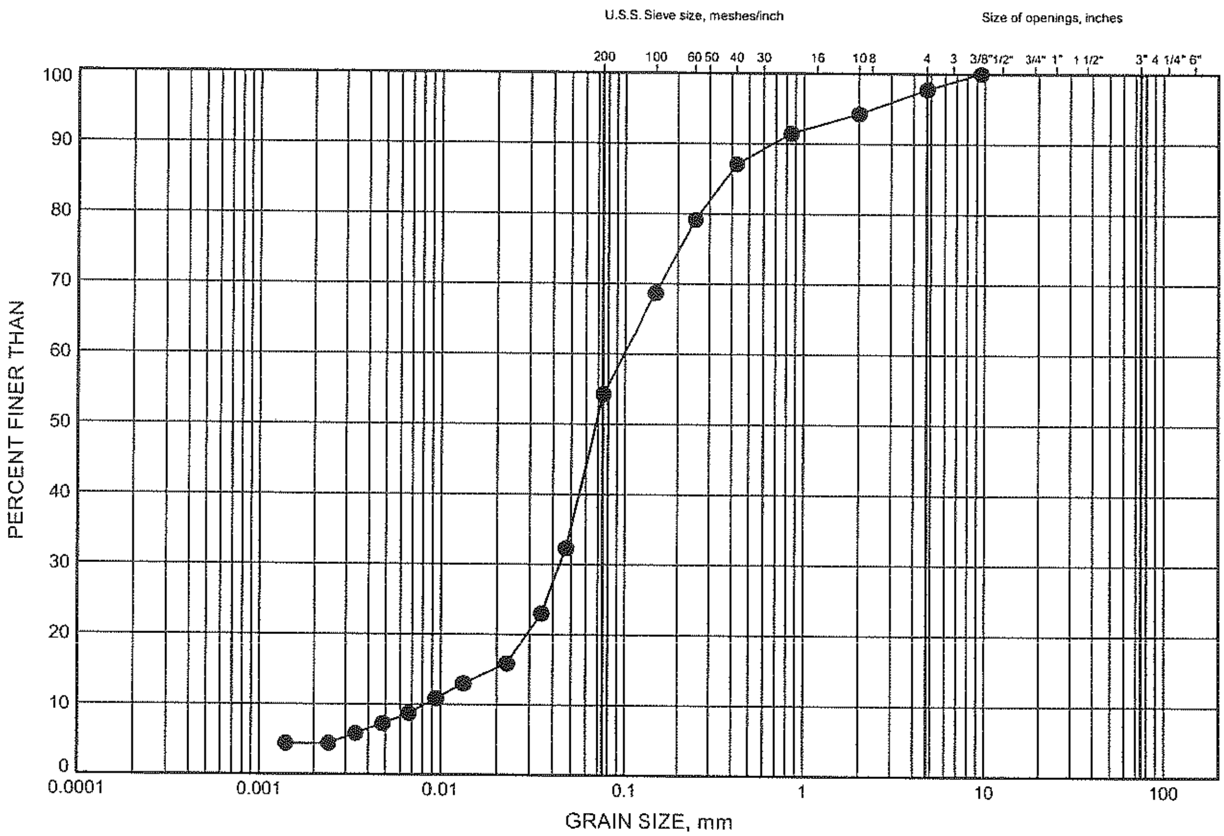
$+^3, \times^3$: Numbers refer to Sensitivity   20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Highway 7 - New

GRAIN SIZE DISTRIBUTION

FIGURE B1

SANDY SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-046	3.35	318.58

W.P.# 408-88-00

Prepared By AN

Checked By RPR

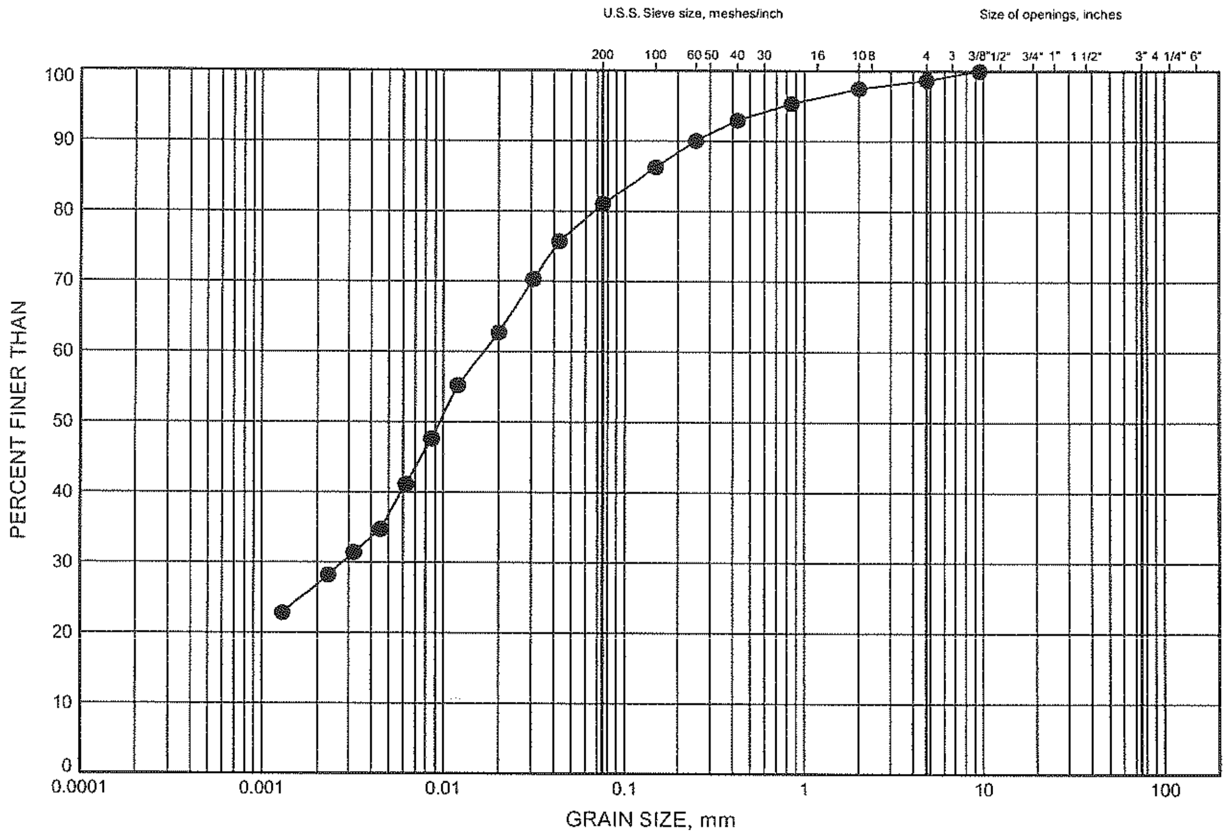
THURBER

Highway 7 - New

GRAIN SIZE DISTRIBUTION

FIGURE B2

CLAYEY SILT TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-046	7.92	314.01

W.P.# 408-88-00

Prepared By AN

Checked By RPR

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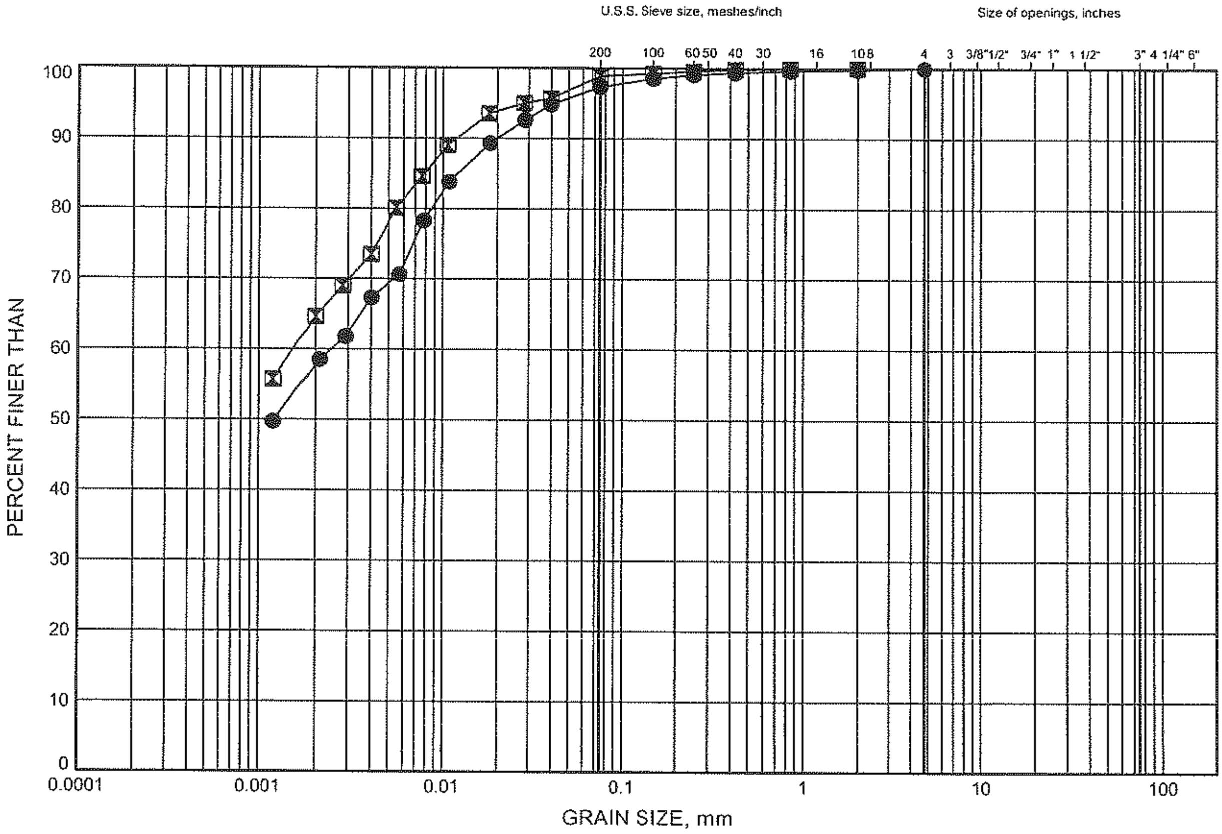
Highway 7 - New
GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY CLAY



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-046	14.02	307.91
⊠	08-046	20.12	301.82

W.P.# 408-88-00

Prepared By AN

Checked By RPR

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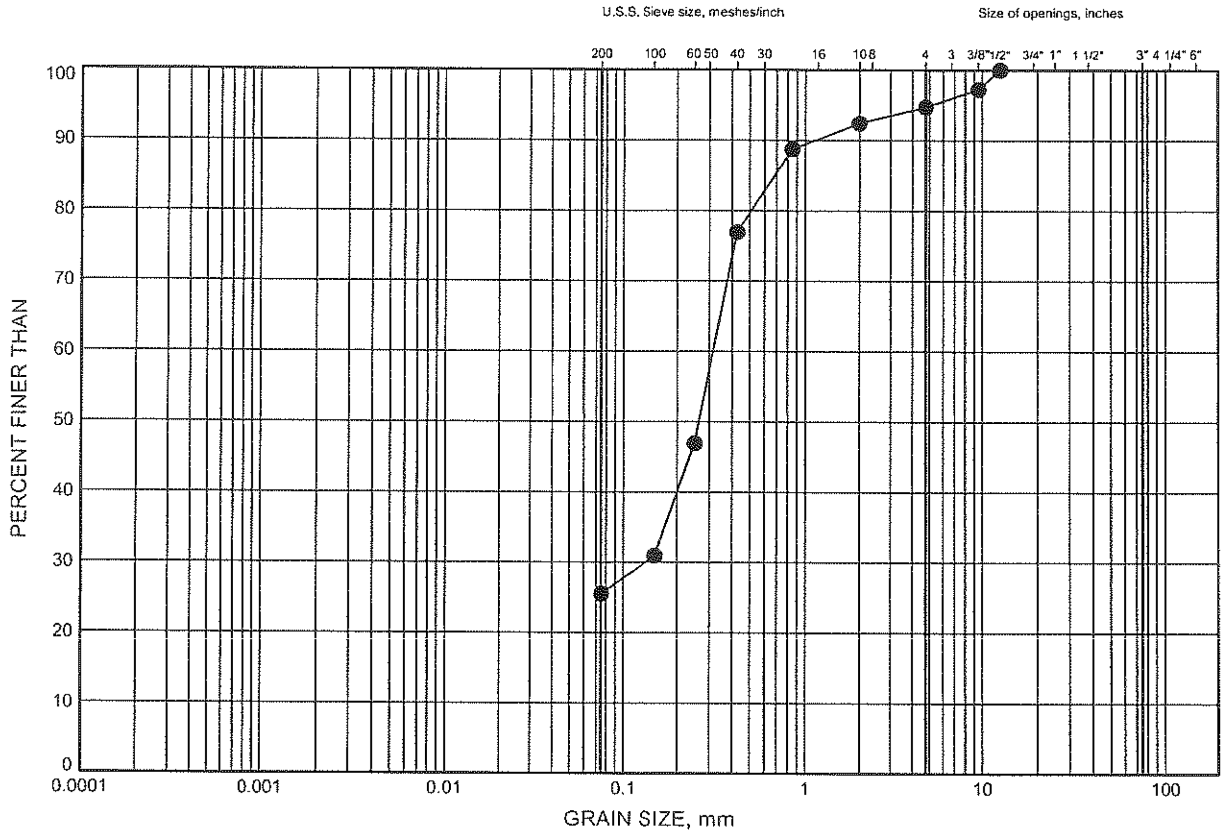
Highway 7 - New

GRAIN SIZE DISTRIBUTION

FIGURE B4

SAND



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-046	29.26	292.67

W.P.# 408-88-00

Prepared By AN

Checked By RPR

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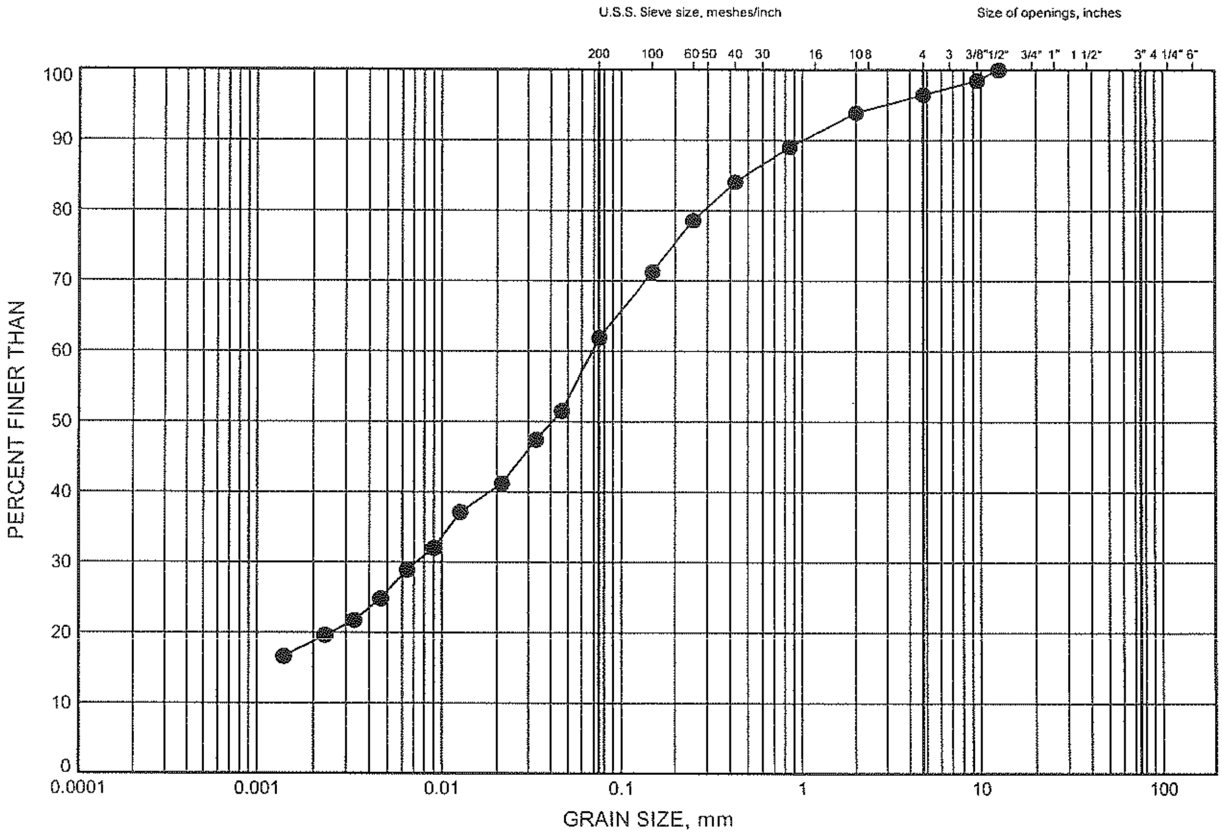
GRAIN SIZE DISTRIBUTION - THURBER 6417R.GPJ 11/24/08

Highway 7 - New
GRAIN SIZE DISTRIBUTION

FIGURE B5

SANDY SILT TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-046	30.78	291.15

GRAIN SIZE DISTRIBUTION - THURBER 6417R.GPJ 11/24/08

W.P.# 408-88-00
Prepared By AN
Checked By RPR

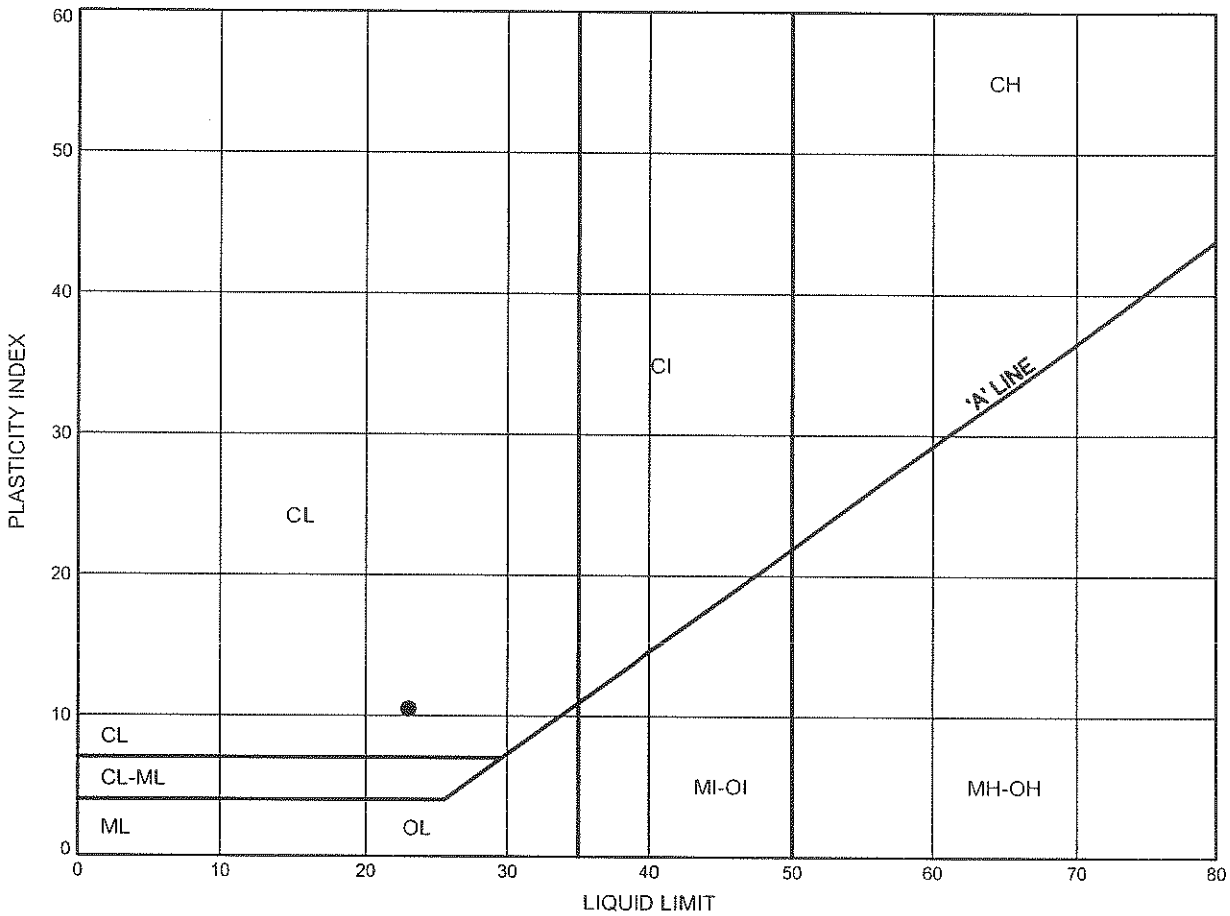
THURBER

Highway 7 - New

ATTERBERG LIMITS TEST RESULTS

FIGURE B6

CLAYEY SILT TILL



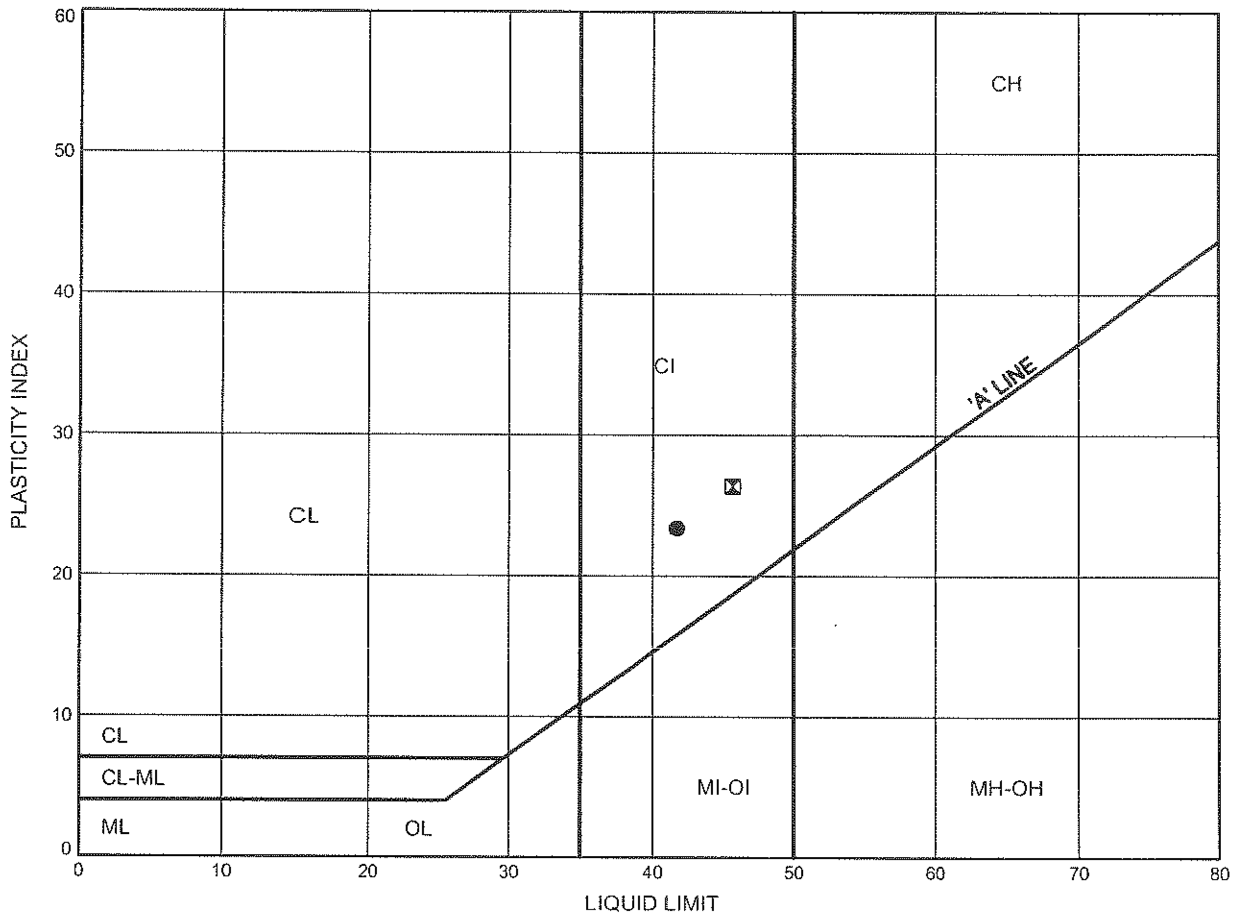
SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-046	7.92	314.01

Highway 7 - New

ATTERBERG LIMITS TEST RESULTS

FIGURE B7

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-046	14.02	307.91
⊠	08-046	20.12	301.82

Date November 2008

Project 408-88-00

THURBER

Prep'd AN

Chkd. RPR

THURBALT 6417R.GPJ 11/24/08

THURBER

Appendix C

Analytical Laboratory Test Results
Present Investigation

FINAL REPORT

CA14437-AUG19 R1

11375 Hwy 7 New, Kitchener

Prepared for

Thurber Engineering Ltd.

FINAL REPORT

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	Thurber Engineering Ltd.	Project Specialist	Rob Irwin B.Sc., C.Chem
		Laboratory	SGS Canada Inc.
Address	103, 2010 Winston Park Drive Oakville, ON L6H 5R7, Canada	Address	185 Concession St., Lakefield ON, K0L 2H0
Contact	Nancy Berg	Telephone	705-652-2361
Telephone	905-829-8666 x 228	Facsimile	705-652-6365
Facsimile		Email	rob.irwin@sgs.com
Email	nberg@thurber.ca	SGS Reference	CA14437-AUG19
Project	11375 Hwy 7 New, Kitchener	Received	08/13/2019
Order Number		Approved	08/19/2019
Samples	Soil (5)	Report Number	CA14437-AUG19 R1
		Date Reported	08/19/2019

COMMENTS

Temperature of Sample upon Receipt: 4 degrees C

Cooling Agent Present: yes

Custody Seal Present: no

Chain of Custody Number: 009972

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

Rob Irwin B.Sc., C.Chem

ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO CHARTERED Robert Irwin CHEMIST

TABLE OF CONTENTS

FINAL REPORT

Client: Thurber Engineering Ltd.
Project: 11375 Hwy 7 New, Kitchener
Project Manager: Nancy Berg
Samplers: Nancy Berg

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

Parameter	Units	RL		5	6	7	8	9
			Sample Name	CN16-10 SS5	CN16-04 SS4	CN16-15 SS4	RW24-02 SS4	NE16-09 SS4
			Sample Matrix	Soil	Soil	Soil	Soil	Soil
			Sample Date	19/07/2019	23/07/2019	18/07/2019	06/08/2019	06/08/2019
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**	**Result**	**Result**
Corrosivity Index								
Corrosivity Index	none	1		4	1	5	11	14
Soil Redox Potential	mV	-		306	312	255	263	227
Sulphide	%	0.02		< 0.02	< 0.02	0.02	< 0.02	< 0.02
pH	pH Units	0.05		8.56	8.29	7.88	8.18	8.66
Resistivity (calculated)	ohms.cm	-9999		5100	3200	2500	780	1400

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

			Sample Number	5	6	7	8	9
			Sample Name	CN16-10 SS5	CN16-04 SS4	CN16-15 SS4	RW24-02 SS4	NE16-09 SS4
			Sample Matrix	Soil	Soil	Soil	Soil	Soil
			Sample Date	19/07/2019	23/07/2019	18/07/2019	06/08/2019	06/08/2019
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**	**Result**	**Result**
General Chemistry								
Conductivity	uS/cm	2		195	317	400	1280	736

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

			Sample Number	5	6	7	8	9
			Sample Name	CN16-10 SS5	CN16-04 SS4	CN16-15 SS4	RW24-02 SS4	NE16-09 SS4
			Sample Matrix	Soil	Soil	Soil	Soil	Soil
			Sample Date	19/07/2019	23/07/2019	18/07/2019	06/08/2019	06/08/2019
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**	**Result**	**Result**
Metals and Inorganics								
Moisture Content	%	0.1		20.1	6.1	24.6	13.1	6.5
Sulphate	µg/g	0.4		25	12	100	31	13

FINAL REPORT

CA14437-AUG19 R1

Client: Thurber Engineering Ltd.

Project: 11375 Hwy 7 New, Kitchener

Project Manager: Nancy Berg

Samplers: Nancy Berg

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

			Sample Number	5	6	7	8	9
			Sample Name	CN16-10 SS5	CN16-04 SS4	CN16-15 SS4	RW24-02 SS4	NE16-09 SS4
			Sample Matrix	Soil	Soil	Soil	Soil	Soil
			Sample Date	19/07/2019	23/07/2019	18/07/2019	06/08/2019	06/08/2019
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**	**Result**	**Result**
Other (ORP)								
Chloride	µg/g	0.4		25	7.8	60	760	430

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	DIO0262-AUG19	µg/g	0.4	<0.4	9	20	93	80	120	98	75	125
Sulphate	DIO0262-AUG19	µg/g	0.4	<0.4	13	20	94	80	120	96	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	ECS0029-AUG19	%	0.02	<0.02	ND	20	110	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	EWL0246-AUG19	uS/cm	2	< 0.002	0	10	100	90	110	NA		

FINAL REPORT

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	EWL0246-AUG19	pH Units	0.05	NA	0		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

No: 009972

- 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

Laboratory Information Section - Lab use only

Received By: Oleg Mozhin
Received Date (mm/dd/yyyy): 8/13/19 (mm\dd\yy)
Received Time: 11:05

Received By (signature):
Custody Seal Present: ☐ no
Custody Seal Intact: ☐ no

Cooling Agent Present: ☑ ice
Temperature Upon Receipt (°C): 9.0, 9.0, 9.0

LAB LIMS #: CA14437-Aug19

REPORT INFORMATION	INVOICE INFORMATION
Company: Thurber Engineering Ltd	☐ (same as Report Information)
Contact: Nancy Berg	Company:
Address: 103 - 2010 Winston Park Dr Oakville On L6H 5R7	Contact:
Phone: 647-633-8417	Address:
Email: nberg@thurber.ca	Phone:
Email:	Email:

PROJECT INFORMATION

Quotation #: P.O. #:
Project #: 11375 Site Location/ID: Hwy 7 New, Kitchener

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: Rush Confirmation ID:

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

REGULATIONS

Regulation 153/04: ☐ Table 1 ☐ Table 2 ☐ Table 3 ☐ Table ___ ☐ R/P/I ☐ I/C/C ☐ A/O
Soil Texture: ☐ Coarse ☐ Medium ☐ Fine

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	PAH☐ ABN☐ SVOC(all)☐	PCB Total ☐ ArocLor ☐	PHC F1-F4 ☐ VOC☐ BTEX ☐ BTEX/F1 ☐ F2-F4☐	VOC☐ BTEX☐ THM☐	Pesticides OC ☐ OP ☐	TCLP M&I ☐ VOC ☐ PCB☐ B(a)P ☐ ABN ☐ Ignit.☐	Water Pkg Gen. ☐ Ext. ☐	Sewer Use:	Corrosivity					COMMENTS:
1	CN16-10 SS5	July 19/19		1	soil											✓					
2	CN 16-04 SS4	July 23/19		1	soil											✓					
3	CN16-15 SS4	July 18/19		1	soil											✓					
4	RW24-02 SS4	Aug 6/19		1	soil											✓					
5	NE16-09 SS4	Aug 7/19		1	soil											✓					
6																					
7																					
8																					
9																					
10																					
11																					
12																					

Observations/Comments/Special Instructions

Sampled By (NAME): Nancy Berg	Signature: N Berg	Date: 08/12/19	(mm/dd/yy)	Pink Copy - Client
Relinquished by (NAME): Nancy Berg	Signature: N Berg	Date: 08/13/19	(mm/dd/yy)	Yellow & White Copy - SGS

Revision #: 1.1
Date of Issue: 04 April, 2018

THURBER

Appendix D

Borehole Locations and Soil Strata Drawing

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

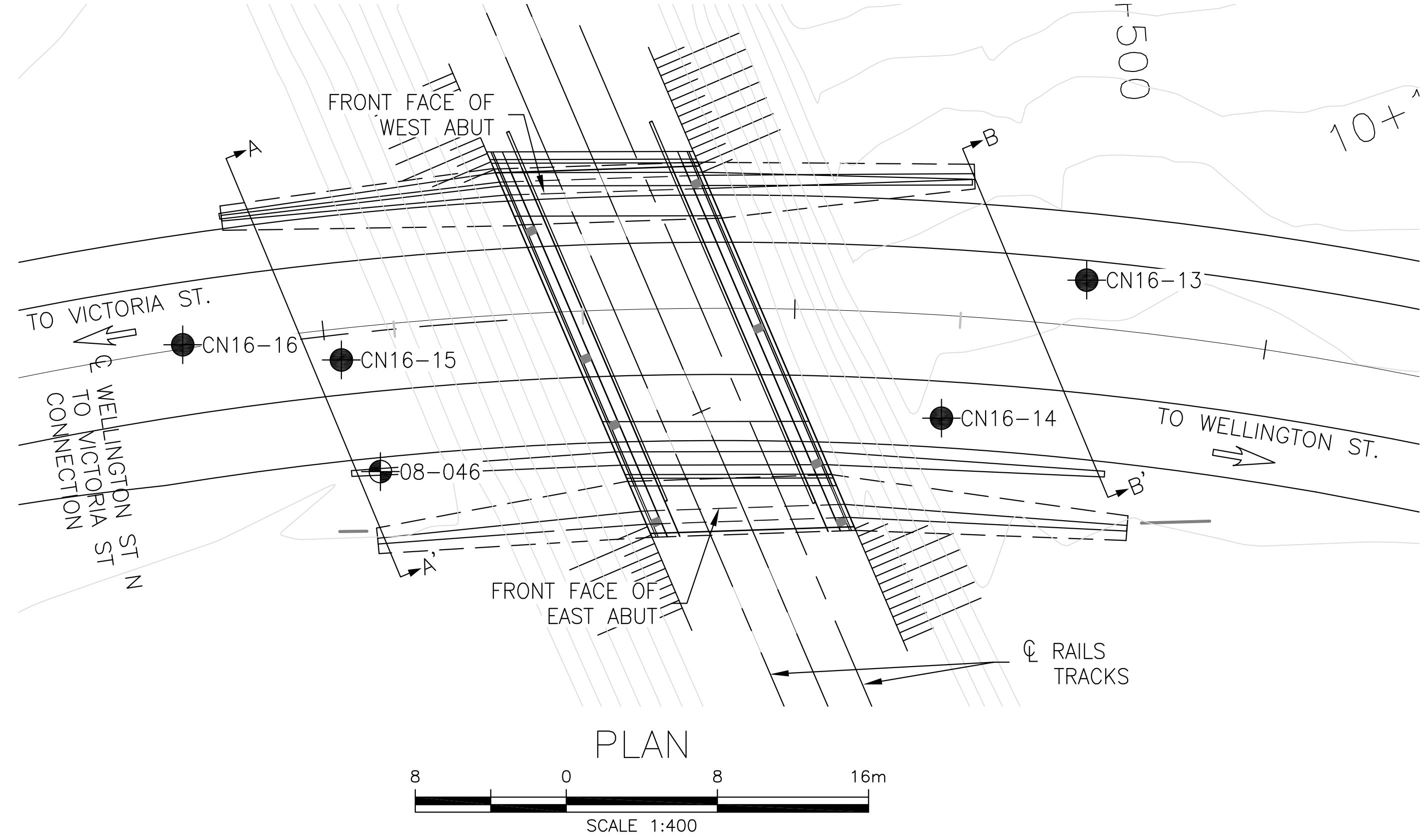
CONT No
GWP No 408–88–00

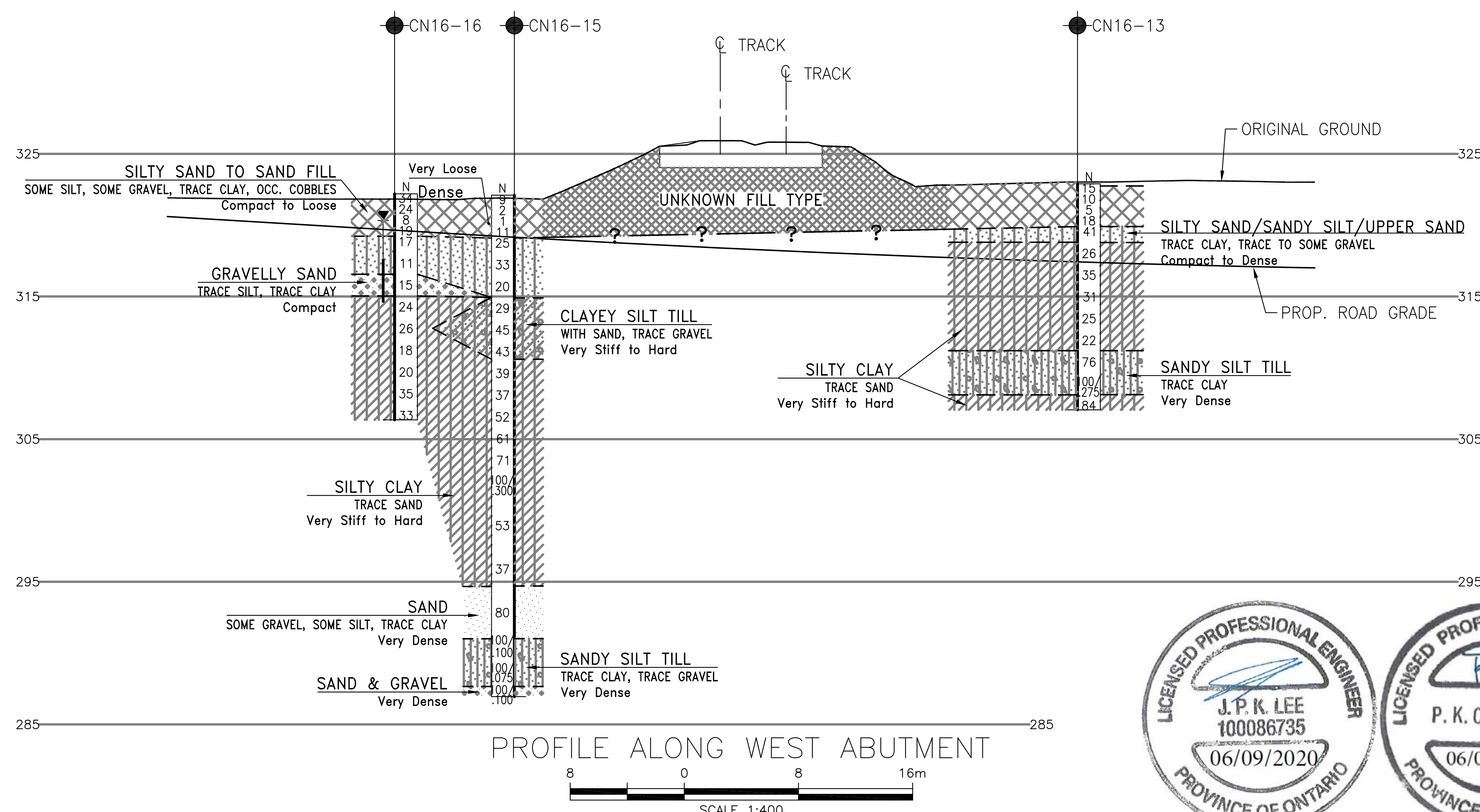
HIGHWAY 7
WELLINGTON ST. TO VICTORIA ST.
CONNECTION BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

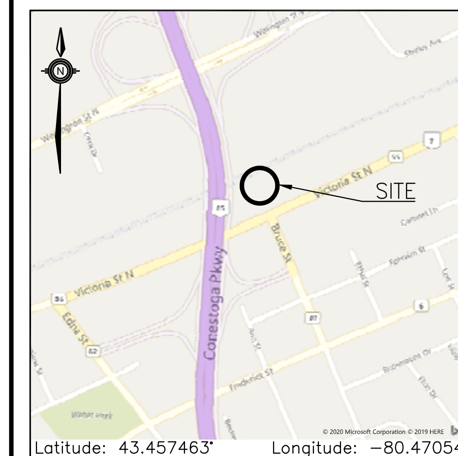
SHEET

WSP

THURBER ENGINEERING LTD.







KEYPLAN

LEGEND

⯐	Borehole (Current Investigation)
⯐	Borehole (Previous Investigation By Thurber)
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
ǀ	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
08–046	321.9	4 814 170.5	226 315.5
CN16–13	322.9	4 814 207.8	226 304.9
CN16–14	323.2	4 814 200.2	226 312.3
CN16–15	322.1	4 814 168.4	226 309.6
CN16–16	322.2	4 814 160.0	226 308.9

-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.

LICENSED PROFESSIONAL ENGINEER
J. P. K. LEE
100086735
06/09/2020
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
06/09/2020
PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION

DESIGN NB	CHK PKC	CODE	LOAD	DATE JUN 2020	
DRAWN AN	CHK NB	SITE	STRUCT	DWG 1	

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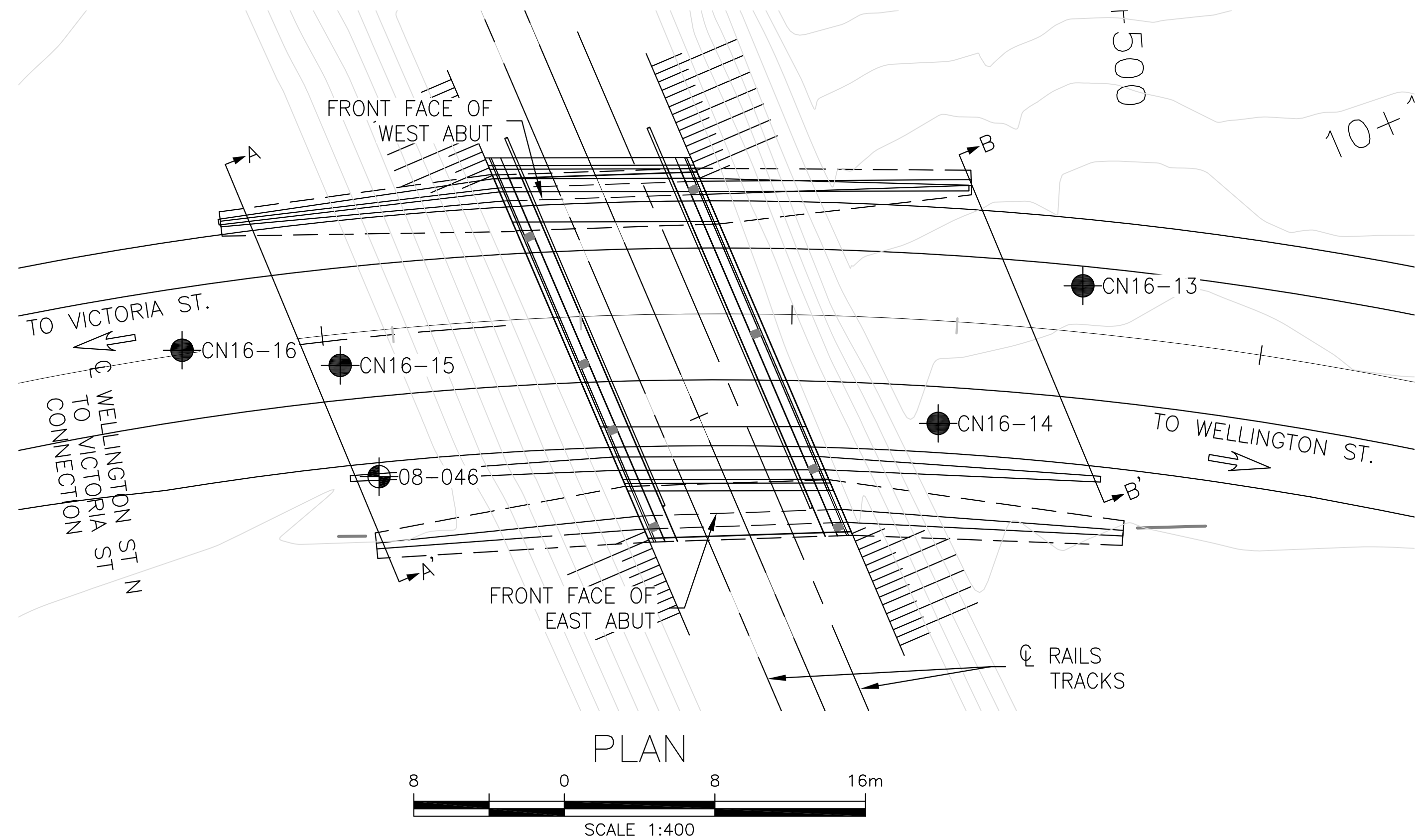
MINISTRY OF TRANSPORTATION, ONTARIO



PLAN

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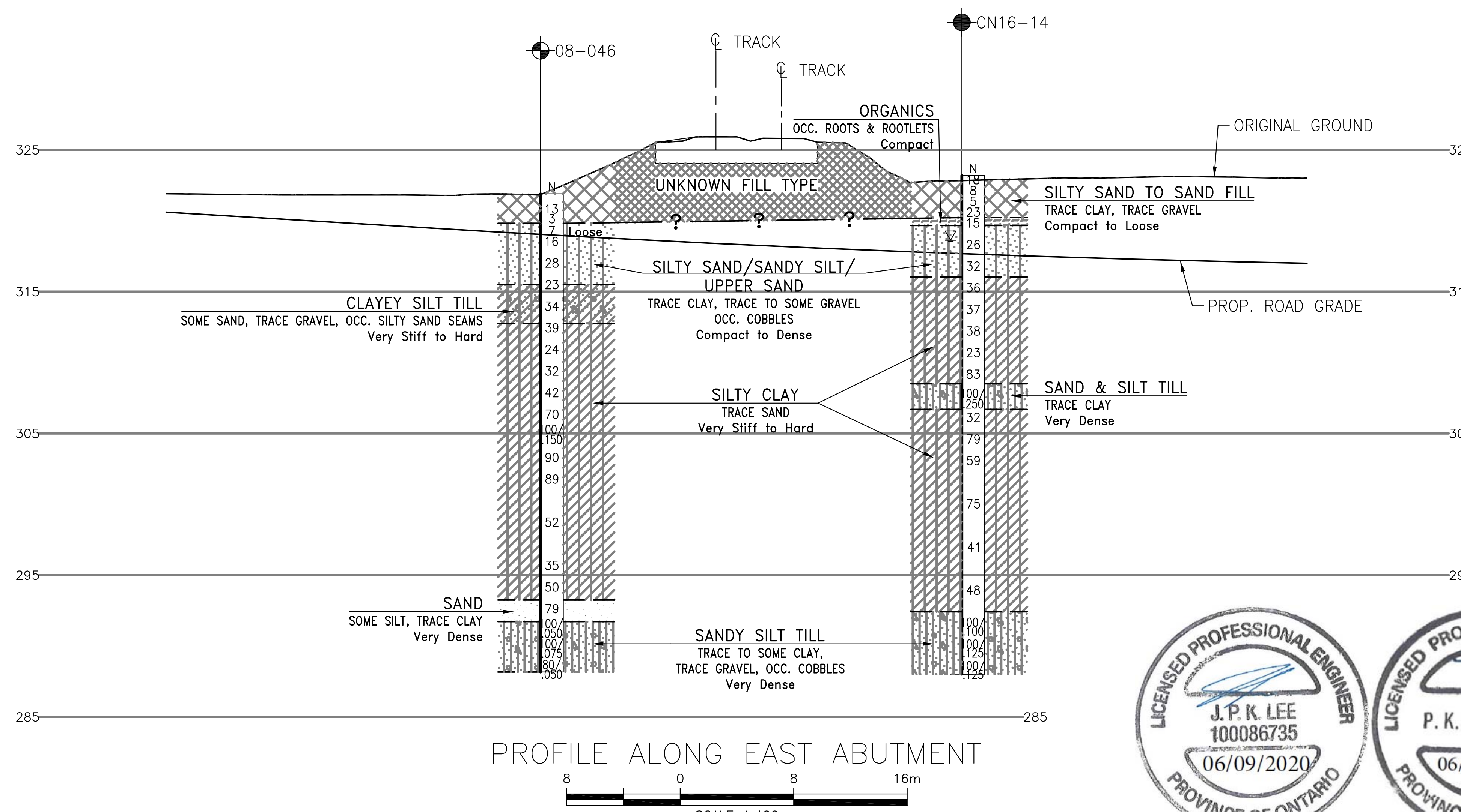

PROFILE ALONG EAST ABUTMENT

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METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No

GWP No 408-88-00

SHEET

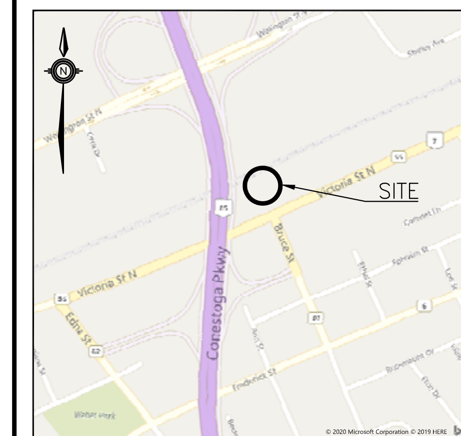
HIGHWAY 7

WELLINGTON ST. TO VICTORIA ST. CONNECTION BRIDGE

BOREHOLE LOCATIONS AND SOIL STRATA

wsp

THURBER ENGINEERING LTD.



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KEYPLAN

LEGEND

⯁	Borehole (Current Investigation)
⯁	Borehole (Previous Investigation By Thurber)
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
08-046	321.9	4 814 170.5	226 315.5
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GEOCRES No.

LICENSED PROFESSIONAL ENGINEER J. P. K. LEE 100086735 06/09/2020 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER P. K. CHATTERJI 06/09/2020 PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION

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DRAWN AN | CHK NB | SITE | STRUCT | DWG 2

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

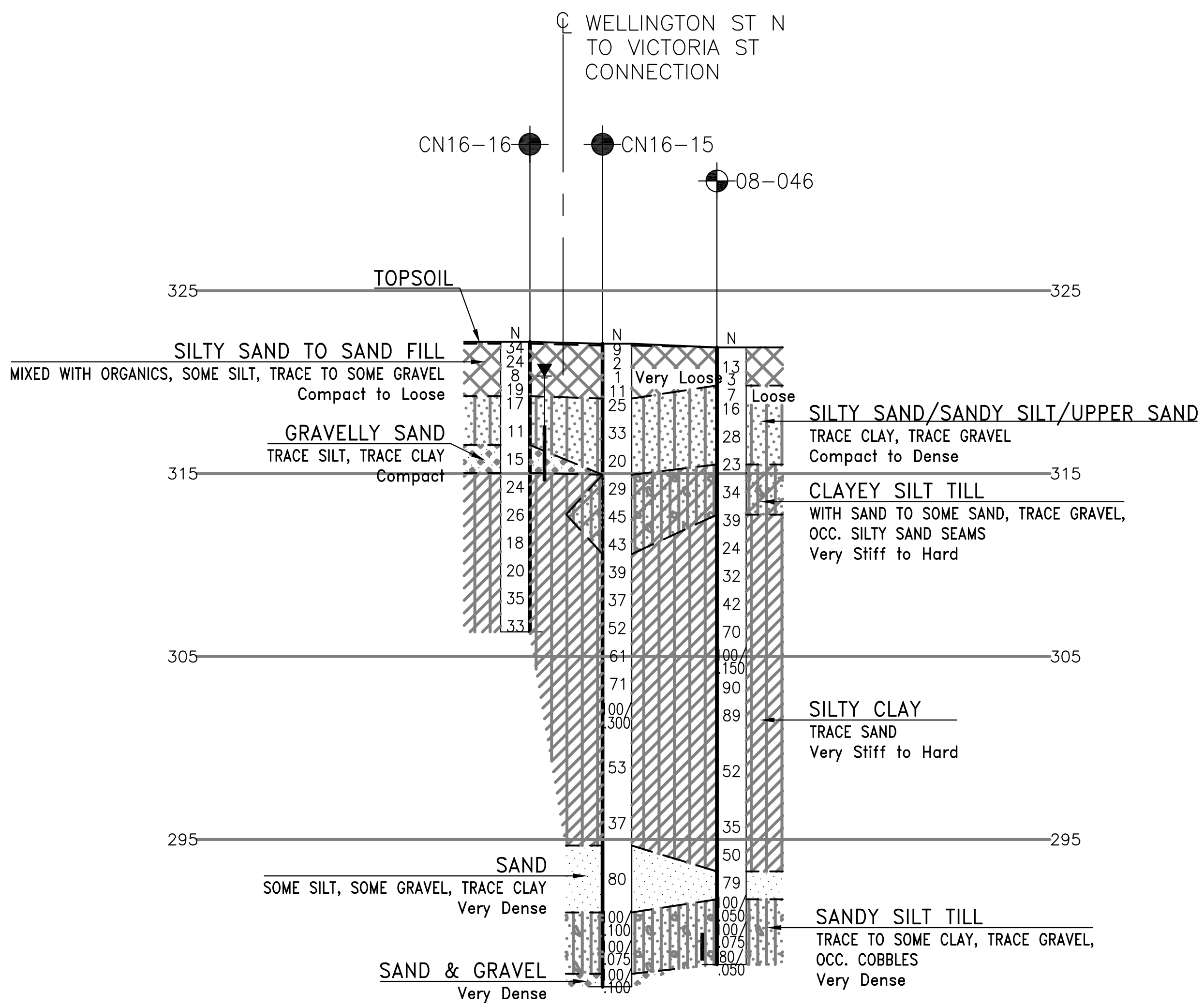
CONT No
GWP No 408-88-00

HIGHWAY 7
WELLINGTON ST. TO VICTORIA ST. CONNECTION BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

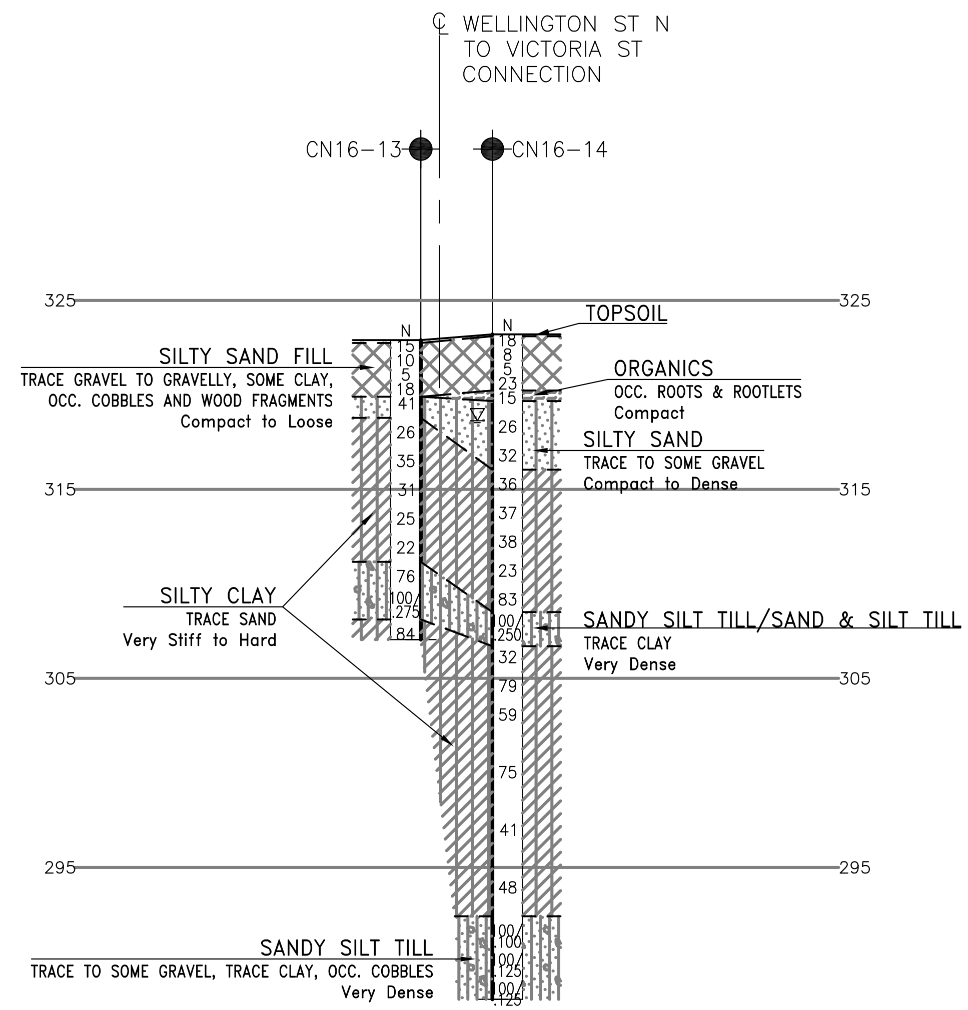
SHEET

wsp

THURBER ENGINEERING LTD.



SECTION A-A'



SECTION B-B'

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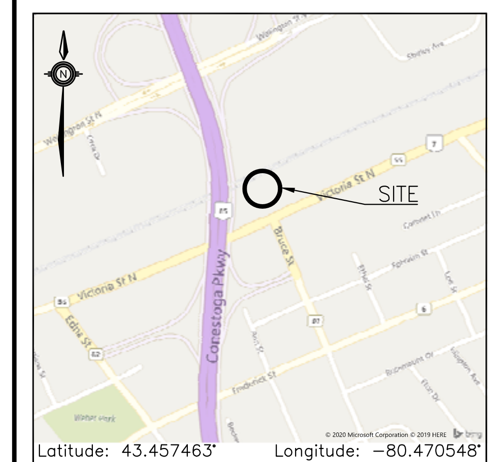

KEYPLAN

LEGEND

⊕	Borehole (Current Investigation)
⊕	Borehole (Previous Investigation By Thurber)
N	Blows /0.3m (Std Pen Test, 475J/blow)
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PH	Pressure, Hydraulic
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	Piezometer
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NO	ELEVATION	NORTHING	EASTING
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GEOCRES No.

REVISIONS	DATE	BY	DESCRIPTION

DESIGN NB	CHK PKC	CODE	LOAD	DATE JUN 2020
DRAWN AN	CHK NB	SITE	STRUCT	DWG 3

FILENAME: H:\Drafting\11000\11375\TED-11375-BHPP-VS.dwg
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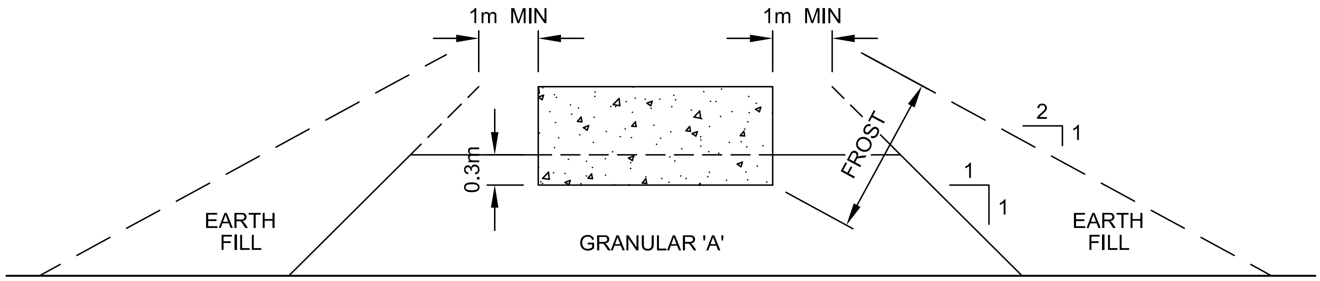
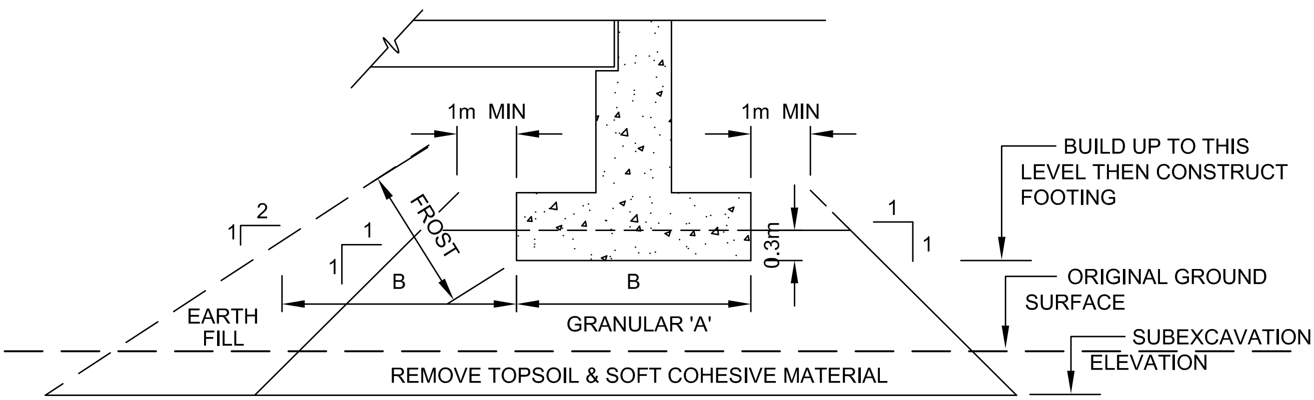
THURBER

Appendix E

Figure
For
Engineered Fill Pad



CROSS-SECTION



LONGITUDINAL SECTION

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S. 501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

ABUTMENT ON COMPACTED FILL
SHOWING GRANULAR 'A' CORE

THURBER ENGINEERING LTD.

ENGINEER :	DRAWN :	APPROVED :
-	MFA	-
DATE : SEPTEMBER 2016	SCALE : N.T.S.	DRAWING No. FIGURE 1

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Appendix F

Foundation Comparison

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COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

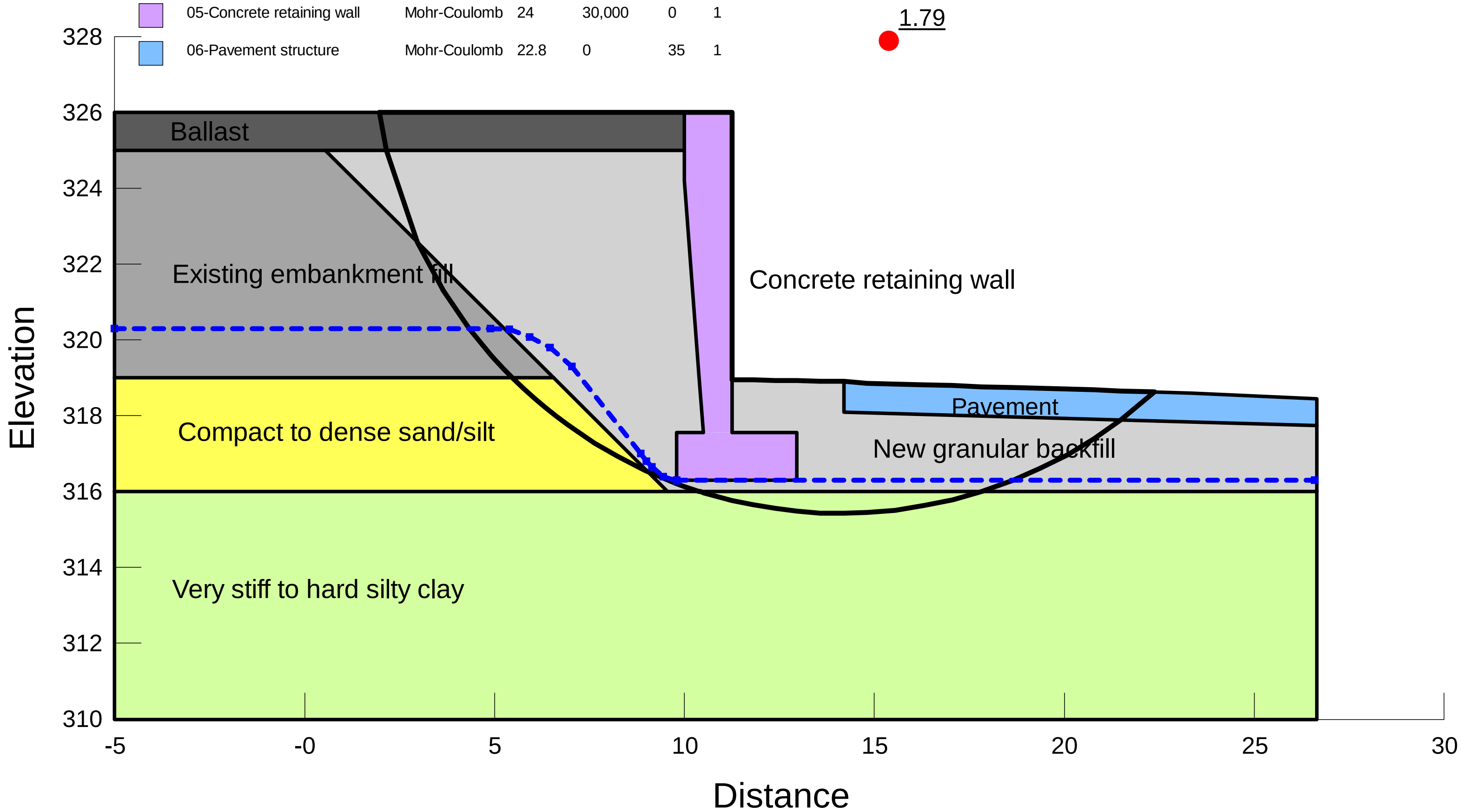
Foundation Element	Spread Footings	Spread Footings on Engineered Fill	Driven Piles	Caisson
Abutments	***Advantages:*** i. Generally less costly construction than deep foundation elements. ***Disadvantages:*** i. Dewatering may be required, depending on depth of excavation. ii. Subexcavation will be required to penetrate fill. **RECOMMENDED (for non-integral abutments)**	***Advantages:*** i. Generally less costly construction than deep foundation elements. ii. Better geotechnical resistance than spread footings on native soils. iii. Founding level can be adjusted. ***Disadvantages:*** i. Excavation of existing fill will be required to place the engineered fill on competent native soils. ii. Dewatering may be required, depending on depth of excavation. **FEASIBLE**	***Advantages:*** i. High geotechnical resistance may be developed by driving the piles into very dense till. ii. Comparatively short abutment stem possible iii. Permits integral abutment design. iv. Readily installed. v. Instalation of piles could continue in freezing conditions. vi. Driven plies require less volume of excavation than footings. ***Disadvantages:*** i. Higher unit cost compared to footings. ii. When driven into hard/very dense till deposits, pipe piles are more prone to pile tip damage in comparison to H-piles. iii. Construction concerns related to the possibility of piles being obstructed by a boulder during driving. **RECOMMENDED (for integral abutments)**	***Advantages:*** i. Construction of caissons could continue in freezing weather. ii. High geotechnical resistance available for units founded on very dense till. iii. Subexcavation of fill and variable material not required. ***Disadvantages:*** i. Higher cost than spread footings ii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons under the water table. iii. Potential difficulty in cleaning and inspecting bases. iv. Installation of deep caissons will be required. **NOT RECOMMENDED**

THURBER

Appendix G

Slope Stability Output

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	00-Ballast	Mohr-Coulomb	24	0	35	1
	01-Existing embankment fill	Mohr-Coulomb	19	0	30	1
	02-New granular fill	Mohr-Coulomb	22	0	32	1
	03- Very stiff to hard silty clay	Mohr-Coulomb	19	0	29	1
	04-Compact to dense sand/silt	Mohr-Coulomb	20	0	32	1
	05-Concrete retaining wall	Mohr-Coulomb	24	30,000	0	1
	06-Pavement structure	Mohr-Coulomb	22.8	0	35	1



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Project
11375 - Hwy 7-New CNR Wellington St. to Victoria St. Connection

Analysis
Concrete Retaining Wall- Drained Analysis

Seismic Coefficient
H: 0g, V: 0g

Last Run
04/28/2020,09:13:23 PM

Scale
1:185

Additional Details

Figure G1

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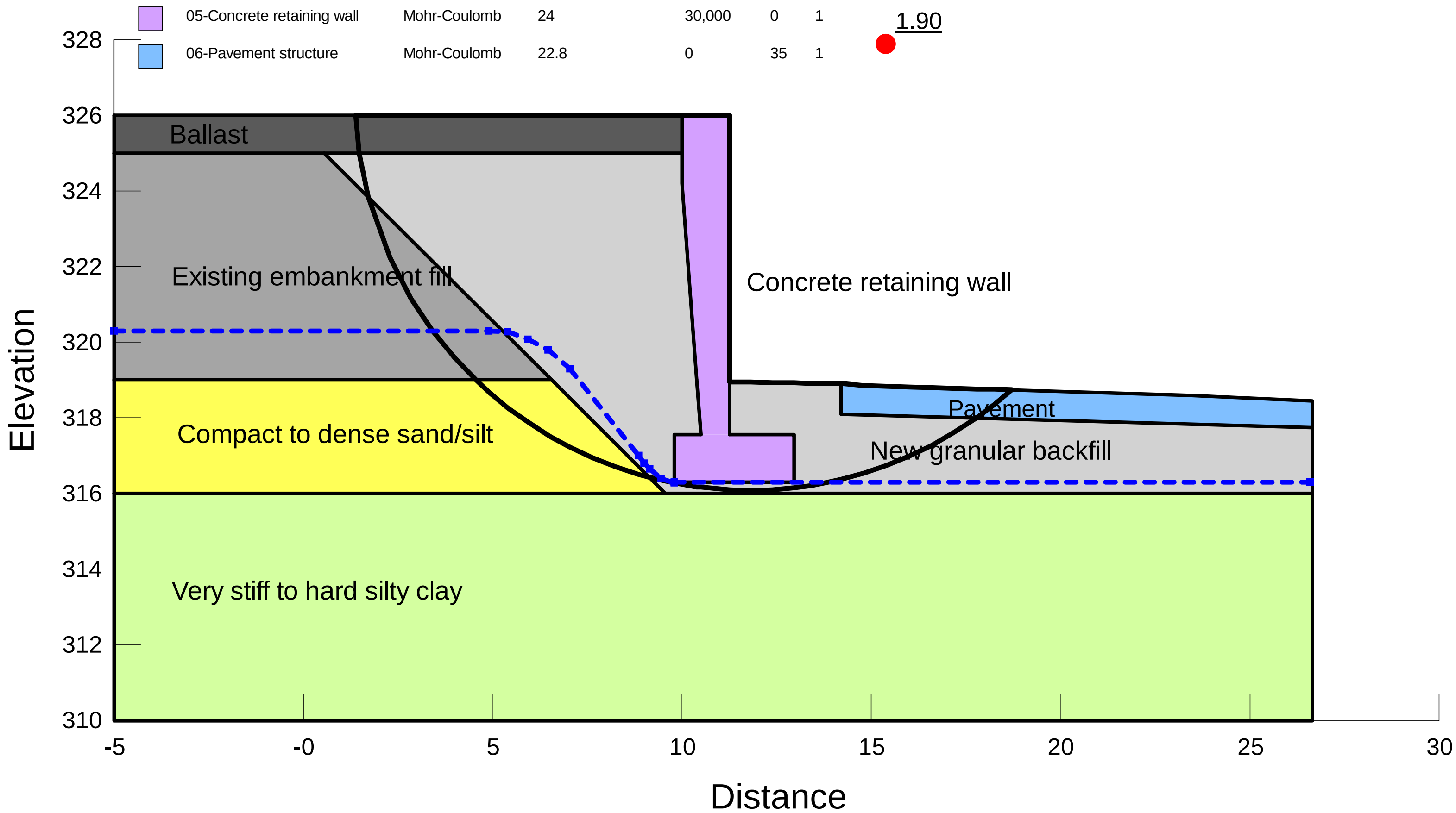
Tool Version: 10.2.1.19666

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	00-Ballast	Mohr-Coulomb	24		0	35	1
	01-Existing embankment fill	Mohr-Coulomb	19		0	30	1
	02-New granular fill	Mohr-Coulomb	22		0	32	1
	03- Very stiff to hard silty clay	Undrained (Phi=0)	19	100			1
	04-Compact to dense sand/silt	Mohr-Coulomb	20		0	32	1
	05-Concrete retaining wall	Mohr-Coulomb	24		30,000	0	1
	06-Pavement structure	Mohr-Coulomb	22.8		0	35	1



Project
11375 - Hwy 7-New CNR Wellington St. to Victoria St. Connection

Analysis
Concrete Retaining Wall- Undrained Analysis

Seismic Coefficient	Last Run	Scale
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Additional Details

THURBER

Figure G2

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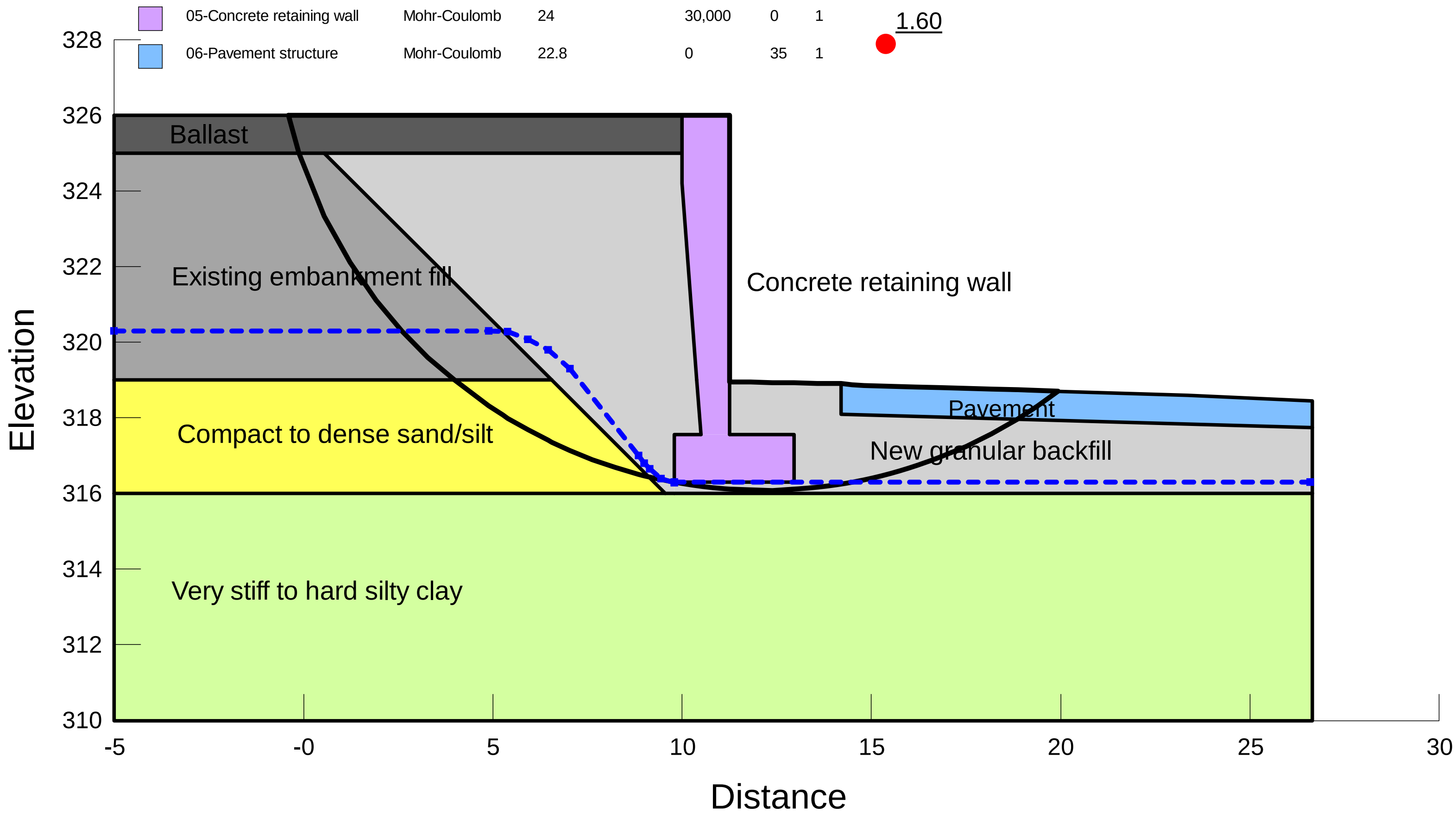
Tool Version: 10.2.1.19666

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	00-Ballast	Mohr-Coulomb	24		0	35	1
	01-Existing embankment fill	Mohr-Coulomb	19		0	30	1
	02-New granular fill	Mohr-Coulomb	22		0	32	1
	03- Very stiff to hard silty clay	Undrained (Phi=0)	19	100			1
	04-Compact to dense sand/silt	Mohr-Coulomb	20		0	32	1
	05-Concrete retaining wall	Mohr-Coulomb	24		30,000	0	1
	06-Pavement structure	Mohr-Coulomb	22.8		0	35	1



THURBER

Project
11375 - Hwy 7-New CNR Wellington St. to Victoria St. Connection

Analysis
Concrete Retaining Wall- Seismic Analysis

Seismic Coefficient	Last Run	Scale
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Additional Details

Figure G3

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Tool Version: 10.2.1.19666

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Appendix H

List of OPSS Documents and NSSP Wording

THURBER

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

-	OPSS PROV 206	Construction specification for grading
-	OPSS PROV 501	Construction specification for compacting
-	OPSS.PROV 517	Construction specification for dewatering
-	SP 517F01	Amendment to OPSS 517
-	SP FOUN0003	Amendment to OPSS.PROV 902
-	OPSS PROV 539	Construction specification for temporary protection systems
-	OPSS PROV 804	Construction specification for seed and cover
-	OPSS PROV 902	Construction specification for excavating and backfilling – Structures
-	SP 109S12	Amendment to OPSS 902
-	OPSS PROV 903	Construction specification for deep foundations
-	SP 109F57	Amendment to OPSS 903
-	OPSS PROV 1010	Material specification for aggregates - base, subbase, select subgrade, and backfill material
-	OPSD 3102.100	Wall abutments, backfill drain
-	OPSD 3101.150	Wall abutment, backfill minimum granular requirement

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2. Suggested text for NSSP on Monitoring of Existing Rail Tracks

Daily visual inspection and settlement monitoring of the rail tracks and rail track embankment must be carried out in the vicinity of the construction works. If any soil loss, track damage or settlement is observed to occur, these matters must immediately be brought to the attention of the Metrolinx CA for determining if further action is required. The Contractor must be prepared to work with Metrolinx to restore the track base and alignment if movement is detected.

3. Suggested text for NSSP on Pile Installation

The presence of cobbles and boulders will potentially have an impact on the installation of piles at the site. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

- The cobbles and boulders may impede the driving of the piles resulting in more arduous driving in the very dense soils.
- Some piles may meet refusal on boulders that are large enough not to be dislodged or broken by the pile driving.
- As a result of the presence of boulders, piles may meet refusal at varying depths.
- Pile driving must be controlled according to the criteria specified for the site.

4. Suggested Text for NSSP on Temporary Protection System and Additional Investigation for Railway Embankment

The presence of obstructions such as cobbles, boulders, railway ties and/or other debris may be encountered during excavation within the railway embankment fill. Boreholes are recommended to be drilled through the railway embankment by the party responsible for the design of the temporary protection/support systems before the design is carried out. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

THURBER

- The cobbles, boulders, railway ties and/or other debris may impede the excavation resulting in more arduous excavation.

5. Suggested Text for NSSP on Groundwater Control

Water seepage due to perched water in the slope, random fill, surface runoff and precipitation should be expected. For temporary excavations at this site, groundwater control will likely be limited to diverting surface runoff and preventing precipitation from entering the excavations supplemented by sump pumping and use of perimeter ditches where required. Filtered sumps must be designed properly so that construction drainage water containing eroded soil and fines do not flow onto the existing roadways. For bridge foundation construction, appropriate dewatering systems must be installed and made operational prior to excavating below the groundwater level. The dewatering scheme 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. It is also important to minimize disturbance of the exposed silty sand surfaces by limiting construction traffic.

The dewatering system is to be designed in accordance with SP FOUN0003 and OPSS.PROV.517. A preconstruction survey is required, thus Designer Fill-In ** in SP FOUN0003 and SP517F01 should be “Yes”. SP FOUN0003 and SP517F01 are attached.

It is recommended that a Professional Engineer with greater than 5 years of experience in designing dewatering systems be retained.

6. Suggested Text for NSSP on Impact on Adjacent Structure

It is critical that Contractor’s excavation and construction activities do not undermine or have any adverse impact on the integrity and performance of the rail tracks, any adjacent structures or underground utilities:

- The lanes of the Kitchener-Waterloo Express way and CN tracks will be open during excavation and foundation construction of the CN bridge over the planned Wellington Street North to Victoria Street Connection

THURBER

- Protection of structure foundations and utilities (if present at this site) during excavation and pile driving.
- Protection of existing approach fills.

7. Suggested Text for NSSP on Impact on Existing Slopes and Cut Slopes

The railway embankment side slopes should be inspected before and after construction for and any surficial disturbance should be documented. Where necessary, remedial measures such as re-vegetation and/or placement of gravel sheeting may be required.

For temporary earth cut, the slopes should be inspected for surficial disturbance.

8. Suggested Text for NSSP on Embankment Construction

No medium to high plastic clays can be used for embankment construction.

9. Suggested Text for NSSP on Environmental Investigation

Soil samples obtained within the cohesionless fill and native cohesionless soils revealed strong gasoline odour. It is recommended that environmental/analytical screening and testing be conducted at this site to determine the quality of the excess excavated soils for soil management purposes (re-use on site and/or off-site disposal). Environmental testing of groundwater should also be conducted for the purpose of PTTW application.