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FOUNDATION INVESTIGATION AND DESIGN REPORT
S-E RAMP CONNECTION BRIDGE UNDER METROLINX TRACKS
HIGHWAY 7-NEW, KITCHENER TO GUELPH
G.W.P. 408-88-00

Geocres Number: 40P8-279

Report

To

WSP

Date: July 17, 2020
File: 11375

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TABLE OF CONTENTS

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Appendices

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of the proposed S-E Ramp Rail Bridge under the Metrolinx tracks in the Regional Municipality of Waterloo. This structure is part of the exit ramp from Kitchener Waterloo Expressway (KWE) northbound lanes (NBL) north to the proposed New Highway 7 for the Highway 7-New Project.

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

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, S-E Ramp Under CNR Tracks, Highway 7-New, Kitchener to Guelph, G.W.P. 408-88-00, Geocres No. 40P8-162, Report to Ministry of Transportation Ontario West Region, File: 15-64-17, dated June 2, 2009. (Reference 1).

- Foundation investigation report for C.N.R. Subway, Kitchener-Waterloo Expressway, District #4 (Hamilton), W.J. 66-F-37, W.P. 636-64, Geocres Number 40P8-45, dated July 4, 1966. (Reference 2).

Records of boreholes from the previous reports are attached in Appendix B for reference.

2. SITE DESCRIPTION

The site lies in the proximity of the Kitchener-Waterloo Expressway (KWE), approximately 20.0 m to the east of the existing KWE and Metrolinx bridge and 100.0 m north of Victoria Street. At this location, the proposed S-E Ramp will pass under the existing twin CNR tracks running east-west. Approximately 160.0 m west of the existing Metrolinx bridge, the double tracks emerge from a Metrolinx yard with a number of tracks as well as a spur line. The Metrolinx yard extends some 980.0 m west, to Lancaster Street East. The site lies within an area of industrial and commercial lands and is generally flat.

A vacant lot is currently situated on the south side of Metrolinx tracks; lands on the north side of 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.

Site photographs, are included in Appendix E and show the general nature of the land surrounding the drilling locations.

3. SITE INVESTIGATION AND FIELD TESTING

A detailed site investigation was carried out from July 3, 2019 to July 22, 2019. Four boreholes, numbered CN16-09 to CN16-12, were drilled near the west and east abutments of the proposed structure. Boreholes CN16-09 to CN16-12 ranged in depth from 15.8 m to 35.2 m (Elevation 304.2 to 283.4). It should be noted that no boreholes were drilled to investigate the railway embankment due to access constraints as well as restrictions imposed by Metrolinx.

A summary of the borehole locations, designations, borehole termination depths and termination elevations for each borehole is provided in Table 3.1. The coordinates and elevations of the boreholes are given on the drawings and on the individual Record of

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

The Record of Borehole sheets for the current investigation boreholes are included in Appendix A, and the Record of Borehole sheets for the previous investigation boreholes are included in Appendix B. The approximate locations of the five boreholes are shown on the attached Borehole Locations and Soil Strata Drawings in Appendix D.

Prior to commencing the site investigation, utility clearances were obtained for all borehole locations. 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 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 at Borehole CN16-11 to permit for longer term monitoring of groundwater levels. The piezometer consisted of 50 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 were carried out in accordance with the requirements of O. Reg. 903 (as amended by O. Reg. 372/07). The piezometer is planned to be decommissioned in the summer of 2020.

Table 3.1 – Borehole Completion Details

Borehole	Ground Surface Elevation	Borehole Depth / Base Elevation (m)	Piezometer Tip Depth / Elevation (m)	Completion Details
CN16-09	321.6	17.4/304.2	No Installation	Borehole backfilled with bentonite holeplug and cuttings to surface.
CN16-10	319.5	32.2/287.3	No Installation	Borehole backfilled with cement and grout, and bentonite holeplug to surface.
CN16-11	318.6	35.2/283.4	35.1/283.6	Piezometer with 3.0 m slotted screen installed with sand filter from 35.1 m to 31.4 m, bentonite pellets from 31.4 m to 30.5 m, grout from 30.5 m to 4.6 m and bentonite holeplug from 4.6 m to ground surface.
CN16-12	318.7	15.8/302.8	No Installation	Borehole backfilled with cement and grout, and bentonite holeplug to surface.
08-045	322.2	32.1/290.1	No Installation	Borehole backfilled with grout 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 gradation analysis (sieve and hydrometer) and Atterberg Limits testing, where appropriate. The results of this testing program are summarized on the Record of Borehole sheets and figures in Appendix A for the current investigation, and Appendix B for the previous investigation.

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 native soil was collected and submitted to SGS Canada Inc., a CALA accredited analytical laboratory in Lakefield, Ontario, for analytical testing of corrosivity parameters. The results of the analytical testing are summarized in this report and 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 Appendix A and Appendix B and on the “Borehole Locations and Soil Strata” drawings included in Appendix D.

An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions. 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 overlying a cohesionless fill layer, a layer of native sand, underlain by clayey silt till, silty clay, silty sand, and silt till.

5.1 Topsoil

A layer of topsoil was encountered surficially in four boreholes drilled at this site, CN16-09 to CN16-12. It was generally dark brown in colour. The thickness of the topsoil layer ranged from 100 mm to 125 mm. The topsoil thickness may vary between the borehole locations and in other areas of the site.

5.2 Asphalt

Asphalt with a thickness of 150 mm was encountered surficially at Borehole 08-045, which was drilled in an existing parking lot.

5.3 Cohesionless Fill

Cohesionless fill was encountered immediately below the topsoil in four boreholes at this site, Boreholes CN16-09 to CN16-12, and encountered below the asphalt at Borehole 08-045. The fill consisted of a layer of gravelly sand fill in Boreholes CN16-09 and 08-045, underlain by generally silty sand fill, which was encountered in all of the boreholes.

The gravelly sand fill contained some silt, trace to some clay, occasional cobbles, and occasional organics. A gasoline odour was also noted in Borehole 08-045. The gravelly sand fill was generally brown to black in colour.

The silty sand fill ranged in composition from sand with some silt to sandy silt, and also contained trace to some gravel, trace to some clay, and occasional organics and rootlets.

The gravelly sand fill in Boreholes CN16-09 and 08-045 was 2.1 m thick, and extended to depths of 2.2 to 2.3 m (Elevation 319.9 to 319.4). The silty sand fill ranged in thickness from 1.3 to 2.9 m, with the lower boundary of this layer encountered at depths ranging from 1.4 m to 4.6 m (Elevation 318.1 to 315.7).

SPT N-values recorded in the gravelly sand fill ranged from 12 to 67 blows for 0.3 m penetration, indicating a compact to very dense relative density. The silty sand fill was very loose to compact, with SPT N-values ranging from 1 to 14 blows for 0.3 m penetration.

The moisture content of samples of the soil ranged from 8 to 11 percent for the gravelly sand fill, and generally ranged from 11 to 25 percent for the silty sand fill. Two samples of the silty sand fill in Boreholes CN16-11 and 08-045, were measured to contain moisture contents of 60 and 114 percent, indicating the presence of organics within the samples.

Three samples of the cohesionless fill underwent laboratory gradation analysis. These results are summarized on the Record of Borehole sheets included in Appendix A and the grain size distribution curves for these samples are plotted on Figure A1 of Appendix A. The results of this testing are summarized as follows:

Soil Particles	Gravelly Sand Fill (%)	Silty Sand Fill (%)
Gravel	30	9 to 20
Sand	47	47 to 51
Silt	18	24 to 38
Clay	5	5 to 6

5.4 Organics

A layer of organics was encountered below the cohesionless fill layer in Borehole CN16-09, at a depth of 4.1 m (Elevation 317.5).

The thickness of the organics layer was 0.8 m, with the lower boundary of this layer encountered at a depth of 4.9 m (Elevation 316.7).

The organics layer was generally black in colour and contained occasional roots and rootlets.

The SPT N-Value recorded in the organic layer was 4 blows for 0.3 m penetration, indicating a loose relative density.

The moisture content from a sample of the organics layer was measured to be 47 percent.

5.5 Sand

A native sand layer was encountered below the coheionless fill in four boreholes at this site, Boreholes CN16-10 to CN16-12 and 08-045. Native sand was encountered below the organics layer in Borehole CN16-09.

The native sand layer was encountered at depths ranging from 1.4 m to 4.9 m (Elevation 318.1 to 315.7), respectively.

The sand layer was brown to grey in colour and contained some silt to silty, trace to some gravel and trace clay, with occasional cobbles encountered in Boreholes CN16-10 and CN16-11.

The thickness of the sand layer ranged from 1.1 m to 3.7 m, with the lower boundary of the sand layer encountered at depths ranging from 4.1 m to 7.2 m (Elevation 315.8 to 312.6).

SPT N-values recorded in the sand ranged from 3 to 46 blows for 0.3 m penetration, indicating a very loose to dense relative density.

Moisture content of samples of the sand generally ranged from 10 percent to 34 percent.

Two samples of the sand underwent laboratory gradation analysis. These results are summarized on the Record of Borehole sheets included in Appendix A and B and the grain size distribution curves for these samples are plotted on Figure A2 and Figure B1. The results of this testing are summarized as follows:

Soil Particles	Sand (%)
Gravel	1 to 16
Sand	72 to 81
Silt and Clay	12 to 18

5.6 Clayey Silt Till

A layer of clayey silt till was encountered below the sand layer in all boreholes at this site, at depths ranging from 4.1 m to 7.2 m (Elevation 315.8 to 312.6).

The clayey silt till was generally grey in colour and contained some sand to sandy and trace gravel. Occasional silty sand seams were encountered in Borehole 08-045.

The thickness of the clayey silt till ranged from 1.5 m to 4.3 m, with the lower boundary encountered at depths ranging from 5.6 m to 10.7 m (Elevation 313.1 to 310.0).

SPT N-values recorded in the clayey silt till ranged from 16 blows to 33 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 24 percent.

Three samples of the clayey silt till underwent laboratory gradation analysis and Atterberg Limits testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets in Appendix A and Appendix B and the grain size distribution curves for these samples are plotted on Figure A3 and Figure B2. The results of the Atterberg Limits tests are plotted on Figure A6 and B5.

Soil Particles	Clayey Silt Till (%)
Gravel	0 to 6
Sand	17 to 23
Silt	49 to 56
Clay	20 to 27

Index Property	(%)
Liquid Limit	22 to 28
Plastic Limit	13 to 16
Plasticity Index	9 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

Silty clay was encountered below the clayey silt till layer in all boreholes at this site, at depths ranging from 5.6 m to 10.7 m (Elevation 313.1 to 310.0).

The silty clay was generally grey in colour and contained trace sand and trace gravel. Occasional cobbles and silt lenses were encountered in Borehole 08-045.

Borehole CN16-09 was terminated in the silty clay layer at a depth of 17.4 m (Elevation 304.2). Borehole CN16-12 was terminated in the silty clay layer at a depth of 15.8 m (Elevation 302.8).

In Boreholes CN16-10, CN16-11 and 08-45, the thickness of the silty clay ranged from 15.7 m to 18.4 m, with the lower boundary encountered at depths ranging from 23.9 m to 29.1 m (Elevation 295.6 to 293.1).

SPT N-values recorded in the silty clay ranged from 13 blows for 0.3 m penetration and 100 blows for 0.275 m penetration, indicating a stiff to hard consistency.

Moisture content of samples of the silty clay generally ranged from 10 percent to 39 percent.

Ten samples of the silty clay underwent laboratory gradation analysis and eight samples underwent Atterberg Limits testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets in Appendix A and Appendix B and the grain size distribution curves for these

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samples are plotted on Figure A4 and Figure B3. The results of the Atterberg Limits tests are plotted on Figure A7 and B6.

Soil Particles	Silty Clay (%)
Gravel	0
Sand	0 to 8
Silt	21 to 51
Clay	47 to 79

Index Property	(%)
Liquid Limit	37 to 63
Plastic Limit	16 to 23
Plasticity Index	20 to 40

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

5.8 Silty Sand

A silty sand layer was encountered below the silty clay in Boreholes CN16-10, CN16-11 and 08-045, at depths ranging from 23.9 m to 29.1 m (Elevation 295.6 to 293.1).

The silty sand was generally grey in colour, and contained trace gravel, trace clay and occasional cobbles. Tri-cone grinding was noted during drilling in this layer.

The thickness of the silty sand layer ranged from 1.1 m to 3.7 m, with the lower boundary encountered at depths ranging from 26.3 m to 30.2 m (Elevation 292.4 to 291.9).

SPT N-values recorded in the silty sand ranged from 59 blows for 0.3 m penetration to 100 blows for 0.275 m penetration, indicating a very dense relative density.

Moisture content of samples of the silty sand generally ranged from 15 percent to 18 percent.

One sample of the silty sand underwent laboratory gradation analysis. These results are summarized on the Record of Borehole sheets included in Appendix A and the grain size distribution curves for these samples are plotted on Figure A5. The results of this testing are summarized as follows:

Soil Particles	Sandy Silt Till (%)
Gravel	7
Sand	64
Silt	24
Clay	5

5.9 Silt Till

A silt till layer was encountered below the silty sand till in Boreholes CN16-10, CN16-11 and 08-045, at depths ranging from 26.3 m to 30.2 m (Elevation 292.4 to 291.9)

Boreholes CN16-10, CN16-11 and 08-045 were terminated in the silt till at the depth of 32.2 m, 35.2 m and 32.1 m, respectively (Elevation 287.3, 283.4 and 290.1).

The silt till was generally grey in colour, and contained some sand to sandy, trace to some clay, trace to some gravel and occasional cobbles. Tricone grinding was noted during drilling in this layer.

The SPT N-value recorded in the silt till ranged from 100 blows for 0.075 m penetration to 100 blows for 0.15 m penetration, indicating a very dense relative density.

Moisture content of samples of the silt till generally ranged from 10 percent to 19 percent.

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

Soil Particles	Silt Till (%)
Gravel	0
Sand	11
Silt	81
Clay	8

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

5.10 Groundwater Conditions

Water levels were observed in the boreholes during and upon completion of drilling.

One standpipe piezometer was installed at this site, in Borehole CN16-11, to monitor water levels after completion of drilling. The water levels measured in the piezometer are summarized in Table 5.1.1, along with the measurements in the open boreholes upon completion of drilling.

Table 5.1.1 – Water Level Measurements

Borehole	Date	Water Level (m)		Comment
		Depth	Elevation	
CN16-09	July 3, 2019	4.9	316.7	Open borehole
CN16-10	July 19, 2019	-	-	Water level upon completion not available due to use of drilling mud.
CN16-11	July 29, 2019	9.0	309.7	Piezometer
CN16-12	July 10, 2019	-	-	Water level upon completion not available due to use of drilling mud.
08-045	Aug 15, 2008	-	-	Water level upon completion not available.

Water level was measured at 3.2 m depth (Elevation 319.0) on October 5, 2008, in a piezometer previously installed at the site.

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

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 native sand from Borehole CN16-10 (depth of 3.4 m) 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
		CN16-10 (SS5 at 3.4 m)
Soil Redox Potential	mV	306
Sulphide	%	< 0.02
pH	pH Units	8.56
Chloride	µg/g	25
Sulphate	µg/g	25
Conductivity	uS/cm	195
Resistivity (calculated)	ohms.cm	5100

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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 investigation.

The coordinates and elevations for the boreholes were provided by WSP.

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

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

Overall supervision of the field program for the investigation was conducted by Dr. Nancy Berg, P.Eng. Interpretation of the data and preparation of the report was carried out by Ms. Judy Mei, EIT, and Dr. Nancy Berg, 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 S-E Ramp under the Metrolinx dual tracks located east of the KWE in the Regional Municipality of Waterloo, Ontario.

The General Arrangement (GA) drawing provided by WSP, indicates that the new CNR bridge over the S-E ramp 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 13.2 m, and the width is 10.0 m. Based on borehole elevations the existing ground surface ranges from approximately Elevations 318.6 to 321.6. The new S-E ramp under the Metrolinx tracks will be constructed in a cut through the Metrolinx embankment and native ground ranging from 8 m to 8.5 m in total depth, and the final grade will be near Elevation 318.0. Metrolinx tracks, within the structure limits, will be at approximate Elevation 326.0 to 326.1.

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 in operation. Track protection will be required for this stage of construction.

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 the 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 FOUNDATIONS

The stratigraphy identified in the geotechnical investigation consisted primarily of topsoil over loose to compact gravelly sand to silty sand and sand fill, overlying native compact to dense silty sand to sand. A deposit of very stiff to hard silty clay till was contacted below the silty sand to sand. Underlying the silty clay till was a layer of silty clay, which overlaid a layer of very dense sandy silt which was intern underlaid by a layer of silt till. The groundwater level is expected to be at Elevation 319.0 based on previous piezometer measurements.

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 piles 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 G.

10.1 Spread Footing on Native Soil

Spread footings bearing on native soil are generally a cost effective form of foundation and are feasible at this site, however deep temporary excavations will be required to construct the footings. According to the GA drawing, the proposed base of the abutment footings is at approximately Elev. 316.5.

The existing fill is not considered suitable for the support of spread footings, and the spread footings should bear on native undisturbed compact sand, or silty sand, or stiff silty clay till. 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.

It is recommended that all new footings be founded at similar elevations as the existing footings of the CNR bridge over the Conestoga Parkway, where possible, such that the latter will not be undermined. It is critical for the designer to have accurate information on footing base elevations and outlines of existing footing footprints to avoid interference between new and existing footings.

Table 10.1 – Geotechnical Resistances for Spread Footings

Foundation Unit	Borehole	Highest Founding Elevation (m)	Founding Stratum	Factored ULS_f (kPa)	Factored SLS_f (up to 25 mm settlement) (kPa)
West Abutment	CN16-10	315.0 (*)	Very Stiff Clayey Silt Till	400	300
East Abutment	CN16-09 CN16-11	314.5	Compact to Dense Sand	400	300

Note (*): Not recommended to found the footing below this elevation due to risk of undermining the existing east abutment footing of Metrolinx bridge structure over the KWE is located in close proximity (i.e. within 5 m) of the west abutment of S-E Ramp Rail bridge

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) Clause 6.10.2 to Clause 6.10.5.

The geotechnical SLS values, as well as the allowable bearing capacity, 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 sand or silty sand may be computed based on an ultimate coefficient of friction, tan δ, of 0.5. 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).

Founding elevations presented in Table 10.1 will be below groundwater level (Elev. 319.0). 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 Foundation Specialist 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.

10.1.1 Construction of Spread Footing at West Abutment

The recommended founding elevation for spread footing at the west abutment is at 315 m, which is at approximately the same founding elevation as the adjacent existing KWE bridge east abutment footing based on the GA drawing. However, prior to finalizing the design, it is imperative to confirm the actual base elevation and extent of the existing footings. Special attention/care should be given to excavation operations in close proximity to the existing footing to avoid undermining the foundation of the existing footing and any damage to the existing structures. If the new structure footing base is planned to be below Elevation 315 m the following method to prevent undermining, settlement or damage of the existing footings and bridge structure should be implemented:

1. Construct the new west abutment footing within a fully supported, shored enclosure. The shoring should be rigid enough not to destabilize the adjacent existing bridge footing and must be installed prior to the start of footing excavation.

Settlement monitoring of the existing east abutment footings should be conducted before, during and after construction.

Measures must be taken during detail design to control undermining, settlement or damage of the existing footings.

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 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 sand/silty sand 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	CN16-10	317.0
East Abutment	CN16-09 CN16-11	314.5

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 Clause 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. 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 Foundation Specialist 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 of ±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 F.

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 base boiling and heave within a layer of water bearing silty sand till 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 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	CN16-10	289.5	26.5	1,500	1,300	1,650	1,450
East Abutment	CN16-11	290.0	26.0	1,500	1,300	1,650	1,450

THURBER

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	CN16-10	289.5	26.5	1,200	1,050	1,400	1,200
East Abutment	CN16-11	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 pile 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.

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, Silty Clay Till (cohesive soils)

$$k_s = 67\ C_u / B \qquad (kN/m^3)$$

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 to Sandy Silt Till (cohesionless soils)

k_s = $n_{h.}\ z / B$ (kN/m^3)

p_{ult} = $3 \cdot \gamma' \cdot z \cdot K_p$ (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	CN16-10	319.5 to 318.0	-	20	2.9	2,200	Loose Silty Sand Fill
		318.0 to 315.5	-	11*	3.0	2,900	Compact Sand
		315.5 to 312.5	180	10*	-	-	Very stiff to Hard Clayey Silt Till
		312.5 to 295.5	200	10*	-	-	Very Stiff to Hard Silty Clay
		295.5 to 292.0	-	11*	3.5	6,000	Very Dense Silty Sand
		292.0 to 287.5	-	11*	3.7	8,000	Very Dense Sandy Silt Till
East Abutment	CN16-09 CN16-11	321.5 to 319.0	-	20	2.9	2,500	Compact Gravelly Sand Fill
		319.0 to 316.5	-	8*	2.7	1,000	Very loose to Loose Silty Sand Fill
		316.5 to 314.0	-	10*	3.0	2,000	Loose to Compact Silty Sand

		314.0 to 311.5	130	10*	-	-	Stiff to Very Stiff Clayey Silt Till
		311.5 to 294.0	200	10*	-	-	Very Stiff to Hard Silty Clay
		294.0 to 292.5	-	11*	3.5	6,000	Very Dense Silty Sand
		292.5 to 283.5	-	11*	3.7	8,000	Very Dense Sandy Silt Till

* Buoyant 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 should be installed in accordance with OPSS 903.

At this site, the piles will have to be driven through very dense silt to sandy 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 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) or equivalent.

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 I. 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 rigid frame 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 silt to sandy silt till.
- For non-integral abutments (e.g. rigid frame structure proposed in the GA drawing), footing is feasible but will require deep excavation, dewatering and

shoring protection of adjacent existing foundation. From a constructability point of view, driven piles is a better option.

11. RETAINING WALLS

The GA drawing indicates that construction of two concrete retaining walls are planned on the east side of the proposed structure. 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)	Height (m)
Southeast	CN16-09	14	1 to 7
Northeast	CN16-11	14	1 to 7

To provide an acceptable foundation performance, the retaining walls must be founded on native compact silty sand/sandy silt/sand. 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

Retaining Wall Location	Borehole	Highest Founding Elevation (m)	Founding Stratum	Factored ULS_f (kPa)	Factored SLS_f (up to 25 mm settlement) (kPa)
Southeast	CN16-09	316.0	Compact silty sand	350	250
Northeast	CN16-11	314.5	Compact to Dense Silty Sand	400	300

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 factored geotechnical SLS values given above are based on an estimated total settlement not exceeding 25 mm. Based on

THURBER

AREMA guidelines, an allowable bearing capacity of 250 kPa and 300 kPa may be used for retaining wall foundation design for the Southeast and Northeast retaining walls respectively.

A 800-mm thick layer of organics was encountered at 4.1 m depth (Elevation 317.5) in Borehole CN16-09. This layer must be removed 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 to sand and/or very stiff to hard clayey silt till. Engineered fill placed under the retaining wall footings 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 native, undisturbed soils may be computed based on an ultimate coefficient of friction, tan δ, 0.35 for the very stiff to hard clayey silt till and 0.45 for the compact to dense silty sand to sand. The sliding resistance of cast-in-place concrete placed on engineered fill may be computed based on an ultimate coefficient of friction, tan δ, of 0.55. 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).

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 spots 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.

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 Wall

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

The global stability of the retaining walls must be analyzed after the final location and detail configurations of the walls are confirmed/finalized.

Global stability analyses were carried for the retaining walls. The analyses were carried out 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 to date 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 H1 to H3 in Appendix H. 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	1.9	H1
Static Undrained	1.9	H2
Seismic = 0.097 g	1.6	H3

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 1.9 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 retaining wall 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 and will induce immediate (elastic) settlement in the underlying compact silty sand and stiff to hard clayey silt till and silty clay.

The immediate settlements were assessed using elastic methods. Based on these analyses, the settlement is estimated to be in the order of 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. The bridge end of the retaining wall near the railway may be subjected to live train loads.

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

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

The factors in Table 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 and retaining walls. Reference may be made to OPSD 3102.100 where appropriate.

13. NORTH/ SOUTH APPROACH - PERMANENT CUT

Permanent earth cuts are required to construct the S-E ramp approaches of the Metrolinx bridge structure at this site. Based on available information and GA drawing, the maximum proposed cut for the S-E Ramp will be approximately 8 m from the top of the railway embankment to the proposed S-E Ramp roadway Elevation. South of the railway embankment, the maximum proposed cut is approximately 4 m to 6 m. Within the zone of the proposed Metrolinx bridge, the base of cut is at approximate Elevation 318.0. It is anticipated that the soils at the base of the cut will consist of compact to dense silty sand/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. Additional borehole drilling must be completed to determine the soil conditions of the railway embankment. The groundwater level is expected to be at Elevation 319.0. Hence the base of the cut will be below the water table. Although not investigated, railway embankment fill typically contains obstructions such as cobbles and boulders and other obstructions.

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

Where space permits, permanent open cut slopes may be formed at inclinations not steeper than 2H : 1V.

The proposed base of cut at S-E Ramp grade will be at Elevation 318.0, which is below the groundwater table observed in the site during present and previous investigations. Perched water might be also observed during excavation within the sand fill and native sand layers.

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 cohesionless soils encountered at this site above the clayey silt till and silty clay deposits (i.e. mostly above Elev. 313) 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 led to a positive frost free 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. Surface runoff and precipitation must be prevented from flowing perpendicularly down any slope surface. Erosion protection measures must be provided as necessary to maintain slope stability.

The embankment surface and the track level and alignment should be monitored throughout and after construction to identify any induced settlement. The Contractor must be prepared to work with Metrolinx to restore the track base and alignment if movement is detected.

Further recommendations for cut and excavation are presented in Section 14.

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 S-E Ramp rail bridge structure, the road connection grade will be near Elevation 318.5 and Metrolinx tracks will be at Elevation 326.0 to 326.1. Currently, at the site, the twin tracks are built in an embankment which is approximately 4.5 m high. It is not anticipated that new fill will be placed to change the slope of the existing railway embankment based on the GA.

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 must be advanced through the railway embankment prior to design of the temporary protection/support systems by the party responsible for this work to obtain sufficient subsurface. Obstructions such as cobbles, boulders, and railway ties may be encountered during excavation within the railway embankment fill. Boreholes must be drilled deep enough to confirm footing base elevation and design pile tip elevation. Embankments constructed using granular material, select subgrade material, or clean earth fill will have stable side slopes at inclinations of up to 2H:1V.

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 the clean earth fill must not contain medium or high plastic clay.

The embankment surface and the track level and alignment should be monitored throughout and after construction to identify any construction induced settlement. The Contractor must be prepared to work with Metrolinx to restore the track base and alignment if track settlement or movement is detected.

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 S-E Ramp Rail Structure. 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 no steeper than 2H:1V.

14.2 Settlement

No settlement is expected since no new fill is expected to be placed on the approach embankments. 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 loose to dense silty sand. All excavations must be carried out in accordance with the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the fills and native soils above the water table may be classed as Type 3; sands and fills below the groundwater level may be classed as Type 4. A layer of organics was contacted below the cohesionless fill in Borehole CN16-09, and this soil layer is classified as Type 4.

Obstructions such as cobbles, ballast and railway ties may be encountered during excavation within the embankment fill. The embankment fill information provided by the borehole investigation is limited and therefore the potential presence of obstructions in the railway embankment 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 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 (Metrolinx Rail Bridge from Wellington Street North to Victoria Street Connection) will be constructed approximately 35 m east of this site. Furthermore, the east abutment of the existing rail bridge over KWE is located in close proximity (i.e. within 5 m) of the west abutment of S-E Ramp Rail bridge. All excavations must be carried out in a manner that avoids destabilising the foundations of the existing/new bridges 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 or clean earth fill 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 should 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

The groundwater level at this site is expected to be at Elevation 319.0 based on previous piezometer measurements The groundwater levels measured in the piezometer and open boreholes ranged from 4.9 m to 9.0 m below the ground surface (Elevations 316.7 and 309.7). 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,

THURBER

sheeted excavation (cofferdam) or vacuum well-points for deeper excavations. The dewatering system must be effective to maintain the water level at a minimum depth of 0.5 m below the final footing/pile cap grade throughout construction.

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/Silty Sand	6.2×10^{-4}
Clayey Silt Till	1×10^{-7}
Silty Clay	1×10^{-8}

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 piling locations 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 will 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 I.

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 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 with tie backs 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, the interaction between the temporary support system and the adjacent existing bridge abutment/foundation, and the risks associated with track movement during excavation under an operating railway. This would be achieved through the following possible construction sequence:

1. Install the shoring wall below the existing twin rail tracks to support them during excavation of permanent cut and/or excavation and bridge construction. Protection of the adjacent existing bridge abutment foundation must be provided prior to shoring wall installation.
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 the twin tracks 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 35 m east of this site. This easterly bridge will accommodate the proposed Metrolinx Rail Bridge from Wellington Street North to Victoria Street Connection.

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

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. 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 composition, 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 320.0 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 (e.g. train loading). 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 includes which consists of loose to compact sand fill overlying upper layers of native compact sand/silty sand, stiff clayey silt till, stiff to hard silty clay, lower layers of very dense silty sand to silty sand 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, bridge abutments and 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 ϕ = 35°, γ = 22.8 kN/m^3	OPSS Granular B Type I ϕ = 32°, γ = 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 should 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 bridge structure and associated works. Protection and/or relocation of utilities may be required. Underground utilities should not be undermined or damaged during new foundation construction.

Settlement monitoring of the existing east abutment of the KWE/Metrolinx structure must be conducted before, during and after excavation and construction of the temporary protection/support systems and new bridge footings which will be in very close proximity of the existing structure. The monitoring of track settlement should be accomplished by means of surface and subsurface settlement monitoring points. Existing east abutment footings of the KWE/Metrolinx structure must not be undermined or damaged at any time.

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

- Carry out pre-construction condition survey including documentation of any existing distress on the existing structure (Metrolinx/KWE Bridge).
- 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.
- Inspection of the existing 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 our post-construction condition survey

21. CORROSION AND SULPHATE ATTACK POTENTIAL

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

- The potential for sulphate attack on concrete foundations from the surrounding native 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 negligible. 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. Footing construction adjacent to the existing east abutment footings of the Metrolinx/KWE Rail Bridge.

Special attention should be paid to the following issues:

a) New footing construction must not undermine the existing bridge footings.

b) A practical and appropriate construction method should be selected for construction of temporary protection/support system for construction of the new footings.

c) Settlement monitoring of the existing bridge footings should be conducted before, during and after construction.

2. 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.

3. 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.

4. 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 may 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. 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.

5. 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.

6. 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 dewatering 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 footing 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. For deeper excavation, 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 whether it is necessary to apply for an MOE Permit to Take Water (PTTW).

6. Environmental Investigation

Soil samples obtained within the cohesionless fill 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. Removal of Organics

The thickness and presence of organic deposit were investigated at the borehole location only. The organics layer encountered at a depth of 4.1 m in borehole CN16-09 near the southeast retaining wall may extend to greater depths or be encountered at other locations beyond the borehole location. Careful inspection is crucial to confirm that the all organics within the footprint of the embankments, proposed retaining wall and bridge foundations and road base in the permanent cut have been excavated prior to construction.

23. CLOSURE

Engineering analysis and preparation of the report were carried out by Dr. Nancy Berg, P.Eng.

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

Thurber Engineering Ltd.

Nancy Berg, P.Eng.

Geotechnical Engineer

Jason Lee, P.Eng.,

Principal/Senior Geotechnical Engineer

P.K. Chatterji, P.Eng.

Review Principal, Designated MTO Contact

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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

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

▼ 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

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RECORD OF BOREHOLE No CN16-09

METRIC

GWP# 408-88-00 LOCATION MTM NAD 83 Zone 10: N 4 814 145.3 E 226 267.2 ORIGINATED BY BL

DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY BH

DATUM Geodetic DATE 2019.07.03 - 2019.07.03 LATITUDE LONGITUDE CHECKED BY JPL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
321.6	GROUND SURFACE					
0.0	TOPSOIL (100mm)					
0.1	Gravelly **SAND**, some silt, trace clay, occasional organics, Compact, Brown, Moist (FILL)	1	SS	29		
		2	SS	12		
		3	SS	24		30 47 18 5
319.4 / 2.2	**SAND**, silty to some silt, trace gravel, Very Loose to Loose, Brown, Moist	4	SS	7		
		5	SS	2		
317.5 / 4.1	**ORGANICS**, occasional roots and rootlets, Loose, Black, Moist					
316.7 / 4.9	Silty **SAND**, Loose to Compact, Brown, Wet	6	SS	4	∇	
		7	SS	22		
314.4 / 7.2	Clayey **SILT**, some sand, trace gravel, Stiff, Grey, Moist (TILL)	8	SS	20		
312.9 / 8.7	Silty **CLAY**, trace sand, Stiff, Grey, Moist	9	SS	13		0 8 47 45

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

ELEVATION SCALE: 321, 320, 319, 318, 317, 316, 315, 314, 313, 312

Continued Next Page

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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No CN16-09

METRIC

GWP# 408-88-00 LOCATION MTM NAD 83 Zone 10: N 4 814 145.3 E 226 267.2 ORIGINATED BY BL

DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY BH

DATUM Geodetic DATE 2019.07.03 - 2019.07.03 LATITUDE LONGITUDE CHECKED BY JPL

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, trace sand Very Stiff to Hard Grey Moist (TILL)				311	
		10	SS	13		
					310	
		11	SS	15	309	
					308	
		12	SS	30		
					307	
		13	SS	15	306	
					305	
		14	SS	60		0 4 45 51
304.2 17.4	END OF BOREHOLE AT 17.4m. BOREHOLE OPEN TO 5.2m. AND WATER LEVEL AT 4.9m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS.					

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

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No CN16-10

METRIC

GWP# 408-88-00	LOCATION MTM NAD 83 Zone 10: N 4 814 145.3 E 226 257.3		ORIGINATED BY BL
DIST ___ HWY 7	BOREHOLE TYPE Hollow Stem Augers/Tricone		COMPILED BY BH
DATUM Geodetic	DATE 2019.08.19 - 2019.08.22	LATITUDE ___ LONGITUDE ___	CHECKED BY JPL

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
319.5	GROUND SURFACE						
0.0	TOPSOIL (100mm)						
0.1	Silty **SAND**, some clay, trace gravel, occasional organics Loose Brown to Black Moist (FILL)	1	SS	8	319		
		2	SS	9			
318.1 / 1.4	**SAND**, some silt, trace clay, trace gravel, occasional cobbles Compact Brown Moist	3	SS	30	318		
		4	SS	20	317		1 81 16 2
		5	SS	12	316		Switch to tricone
315.4 / 4.1	Clayey **SILT**, sandy, trace gravel Very Stiff to Hard Grey Moist (TILL)	6	SS	27	315		6 23 51 20
		7	SS	33	314 / 313		
312.3 / 7.2	Silty **CLAY**, trace sand Very Stiff to Hard Grey Moist	8	SS	36	312		
		9	SS	25	311 / 310		

Column headings: 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 CN16-10

METRIC

GWP#	408-88-00	LOCATION	MTM NAD 83 Zone 10: N 4 814 145.3 E 226 257.3	ORIGINATED BY	BL
DIST	HWY 7	BOREHOLE TYPE	Hollow Stem Augers/Tricone	COMPILED BY	BH
DATUM	Geodetic	DATE	2019.08.19 - 2019.08.22 LATITUDE LONGITUDE	CHECKED BY	JPL

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
	Silty **CLAY**, trace sand Very Stiff to Hard Grey Moist				309				
		10	SS	23					
					308				
		11	SS	38	307				
					306				
	Sandy SILT layer (125mm)	12	SS	60		0	2	51	47
					305				
		13	SS	73	304				
					303				
		14	SS	43					
					302				
		15	SS	40	301	0	5	38	57
					300				

Continued Next Page

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

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

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

RECORD OF BOREHOLE No CN16-10

METRIC

GWP# 408-88-00 | LOCATION MTM NAD 83 Zone 10: N 4 814 145.3 E 226 257.3 | ORIGINATED BY BL

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

DATUM Geodetic | DATE 2019.08.19 - 2019.08.22 | LATITUDE ___ LONGITUDE ___ | CHECKED BY JPL

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 Grey Moist	16	SS	59		~21	
					299		
					298		Tricone grinding
					297		
		17	SS	36		~26	
					296		
295.6 23.9	Silty **SAND**, trace gravel, trace clay, occasional cobbles Very Dense Grey Moist						
					295		Tricone grinding
					294		
		18	SS	95		~18	7 64 24 5
					293		
					292		Tricone grinding
291.9 27.6	**SILT**, sandy to some sand, trace to some gravel, trace clay Very Dense Grey Moist (TILL)						
					291		
		19	SS	100/ 0.125		~18	
					290		

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

15 ⊕ 5 (20, 10) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No CN16-10

METRIC

GWP# 408-88-00 LOCATION MTM NAD 83 Zone 10: N 4 814 145.3 E 226 257.3 ORIGINATED BY BL

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

DATUM Geodetic DATE 2019.08.19 - 2019.08.22 LATITUDE LONGITUDE CHECKED BY JPL

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, × 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	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
							289				
			20	SS	100/ 0.100				o		
							288				
287.3			21	SS	100/ 0.150				o		
32.2	END OF BOREHOLE AT 32.2m. CAVED-IN DEPTH AND WATER LEVEL NOT AVAILABLE DUE TO USE OF MUD ROTARY DRILLING. BOREHOLE BACKFILLED WITH CEMENT AND GROUT, THEN HOLEPLUG TO SURFACE.										

ONTMT4S2 MTO-11375(GINTDATA).GPJ 2017TEMPLATE(MTO).GDT 7/9/20

+ 3 , × 3 : Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No CN16-11

METRIC

GWP# 408-88-00 LOCATION MTM NAD 83 Zone 10: N 4 814 188.4 E 226 266.1 ORIGINATED BY BL

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

DATUM Geodetic DATE 2019.07.10 - 2019.07.10 LATITUDE LONGITUDE CHECKED BY JPL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
318.6	GROUND SURFACE						
0.0	TOPSOIL (125mm)						
0.1	Silty **SAND**, some gravel to gravelly, trace clay, occasional organics and rootlets; Very Loose to Compact; Brown; Moist; (FILL)	1	SS	14	318	~60	
		2	SS	12		~23	20 51 24 5
		3	SS	2	317	~14	
316.4	0.6m thick Clayey SILT layer at 2.3m						
2.3	Silty **SAND**, trace gravel, trace clay, occasional cobbles; Very Loose to Loose; Brown; Moist	4	SS	3	316	~27	
		5	SS	5		~28	Switch to tricone
		6	SS	7	315	~20	
	Dense	7	SS	41	314		No recovery (SS7)
		8	SS	46	313	~20	
312.6 / 6.0	Clayey **SILT**, sandy, trace gravel; Hard; Grey; Moist; (TILL)	9	SS	32	312	~13 (wp–wL plotted)	3 21 49 27
		10	SS	30	311	~24	
310.0 / 8.7	Silty **CLAY**, trace sand; Hard; Grey; Moist				310		
	0.5m sandy SILT layer at 9.5m	11	SS	40	309	~10, ~15	

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL.

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $15\oplus5$ (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No CN16-11

METRIC

GWP# 408-88-00 | LOCATION MTM NAD 83 Zone 10: N 4 814 188.4 E 226 266.1 | ORIGINATED BY BL

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

DATUM Geodetic | DATE 2019.07.10 - 2019.07.10 | LATITUDE ____ LONGITUDE ____ | CHECKED BY JPL

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 Moist									
					308					
		12	SS	50						
					307					
		13	SS	60			0	5	48	47
					306					
					305					
		14	SS	45						
					304					
		15	SS	66						
					303					
					302					
		16	SS	65						
					301					
		17	SS	55						
					300					
					299					

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

Continued Next Page

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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No CN16-11

METRIC

GWP# 408-88-00 LOCATION MTM NAD 83 Zone 10: N 4 814 188.4 E 226 266.1 ORIGINATED BY BL

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

DATUM Geodetic DATE 2019.07.10 - 2019.07.10 LATITUDE LONGITUDE CHECKED BY JPL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page					
	Silty **CLAY**, trace sand Hard Grey Moist	18	SS	55		0 1 36 63
					298	
					297	
					296	
		19	SS	33		
					295	
294.2 24.4	Silty **SAND**, some clay, trace gravel Very Dense Grey Moist to Wet				294	
					293	
292.4 26.3	**SILT**, some sand, some clay, trace gravel, occasional cobbles Very Dense Grey Moist (TILL)	20	SS	59		
					292	
					291	Tricone grinding
					290	
		21	SS	100/ 0.0100		
					289	

Continued Next Page

SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; STRAT PLOT

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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No CN16-11

METRIC

GWP# 408-88-00 LOCATION MTM NAD 83 Zone 10: N 4 814 188.4 E 226 266.1 ORIGINATED BY BL

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

DATUM Geodetic DATE 2019.07.10 - 2019.07.10 LATITUDE LONGITUDE CHECKED BY JPL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	Continued From Previous Page						
	SILT, some sand, some clay, trace gravel, occasional cobbles Very Dense Grey Moist (TILL)				288		Tricone grinding
					287		
		22	SS	100/ 0.125		o	
					286		
		23	SS	100/ 0.150	285	o	
					284		
283.4		24	SS	100/ 0.150		o	
35.2	END OF BOREHOLE AT 35.2m. Piezometer installation consists of 50mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.						

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2019.08.11	1.6	317.1
2019.08.29	9.0	309.7

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; 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; GR SA SI CL

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No CN16-12

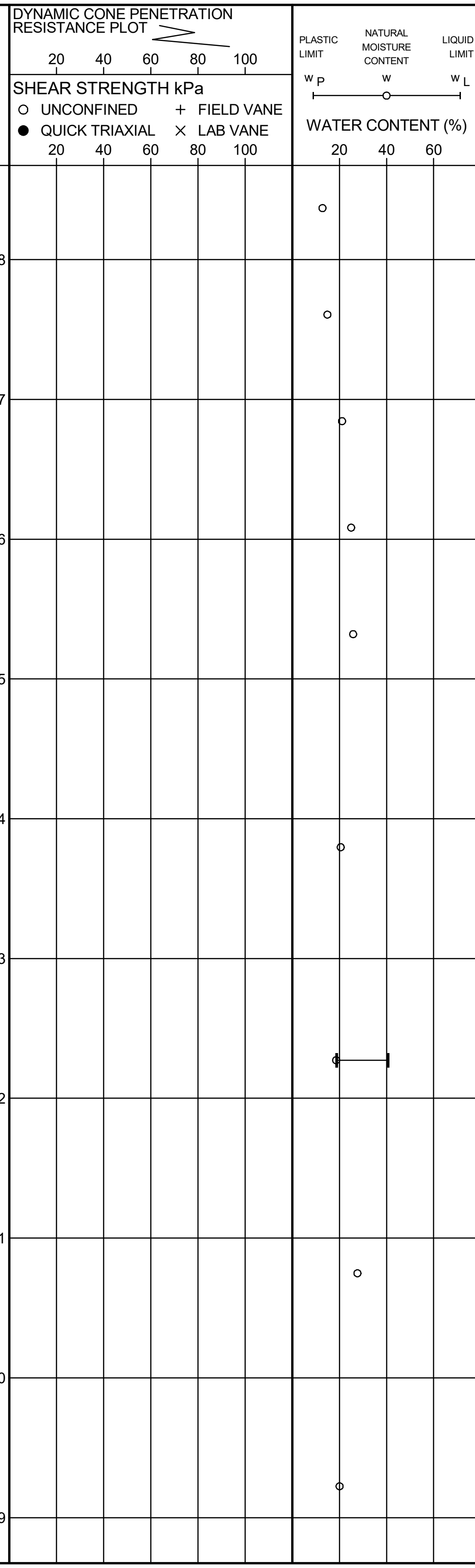
METRIC

GWP# 408-88-00 | LOCATION MTM NAD 83 Zone 10: N 4 814 194.5 E 226 265.5 | ORIGINATED BY BL

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

DATUM Geodetic | DATE 2019.07.10 - 2019.07.10 | LATITUDE ____ LONGITUDE ____ | CHECKED BY JPL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
318.7	GROUND SURFACE				
0.0	TOPSOIL (125mm)				
0.1	Silty **SAND** to **SAND** and **SILT**, trace gravel, trace clay, occasional organics Loose Black/Brown Moist (FILL)	1	SS	6	
		2	SS	7	
		3	SS	9	
	Occasional decayed wood fragments	4	SS	6	9 47 38 6
315.7 3.0	Silty **SAND**, trace gravel, trace clay Very Loose Grey Moist	5	SS	3	Switch to tricone
314.6 4.1	Clayey **SILT**, some sand, trace gravel Very Stiff Grey Moist (TILL)				Tricone grinding
		6	SS	29	
313.1 5.6	Silty **CLAY**, trace sand Very Stiff to Hard Grey Moist				
		7	SS	35	0 5 32 63
		8	SS	21	
		9	SS	39	



Continued Next Page

+3, ×3: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No CN16-12

METRIC

GWP# 408-88-00 | LOCATION MTM NAD 83 Zone 10: N 4 814 194.5 E 226 265.5 | ORIGINATED BY BL

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

DATUM Geodetic | DATE 2019.07.10 - 2019.07.10 | LATITUDE ____ | LONGITUDE ____ | CHECKED BY JPL

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						308	
			10	SS	63		307	
			11	SS	100/ 0.275		306	
							305	
			12	SS	35		304	0 3 41 56
302.8			13	SS	63		303	
15.8	END OF BOREHOLE AT 15.8m. CAVED-IN DEPTH AND WATER LEVEL NOT AVAILABLE DUE TO USE OF MUD ROTARY DRILLING. BOREHOLE BACKFILLED WITH CEMENT AND GROUT, THEN BENTONITE HOLEPLUG TO SURFACE.							

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

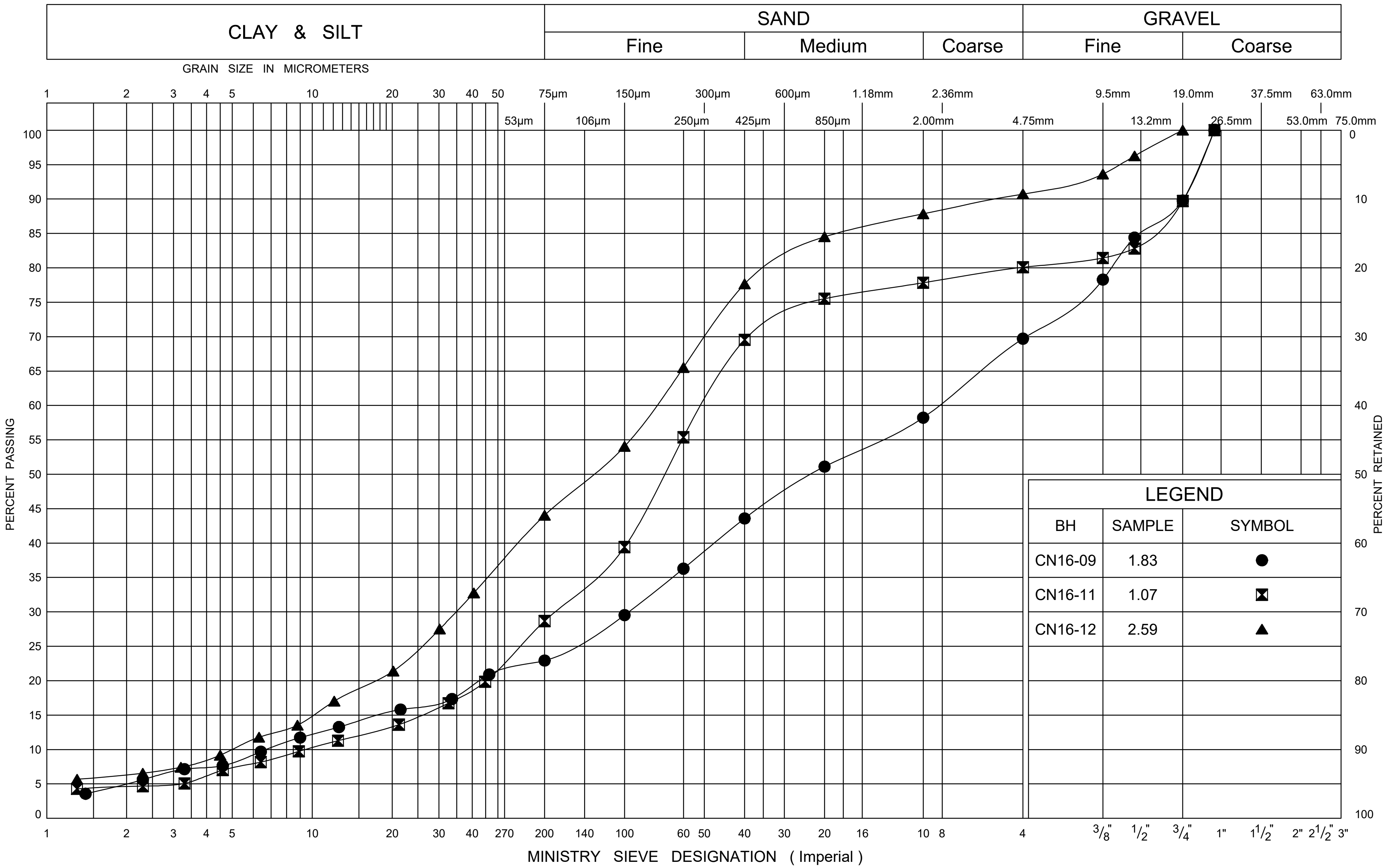
WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

+^{3}, ×^{3}: Numbers refer to Sensitivity

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

78 12 M



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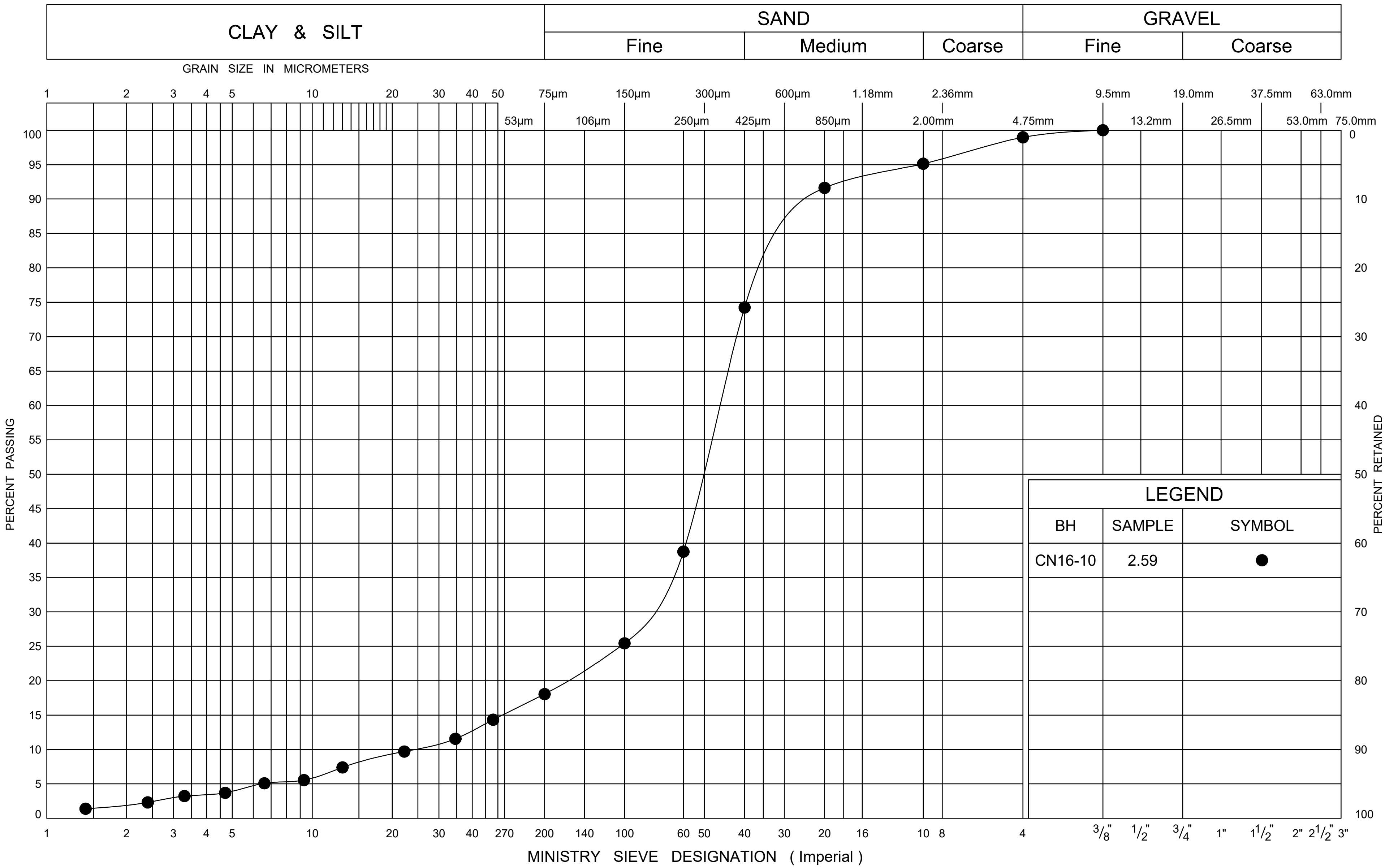
GRAIN SIZE DISTRIBUTION

FILL

FIG No A1

W P 408-88-00

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



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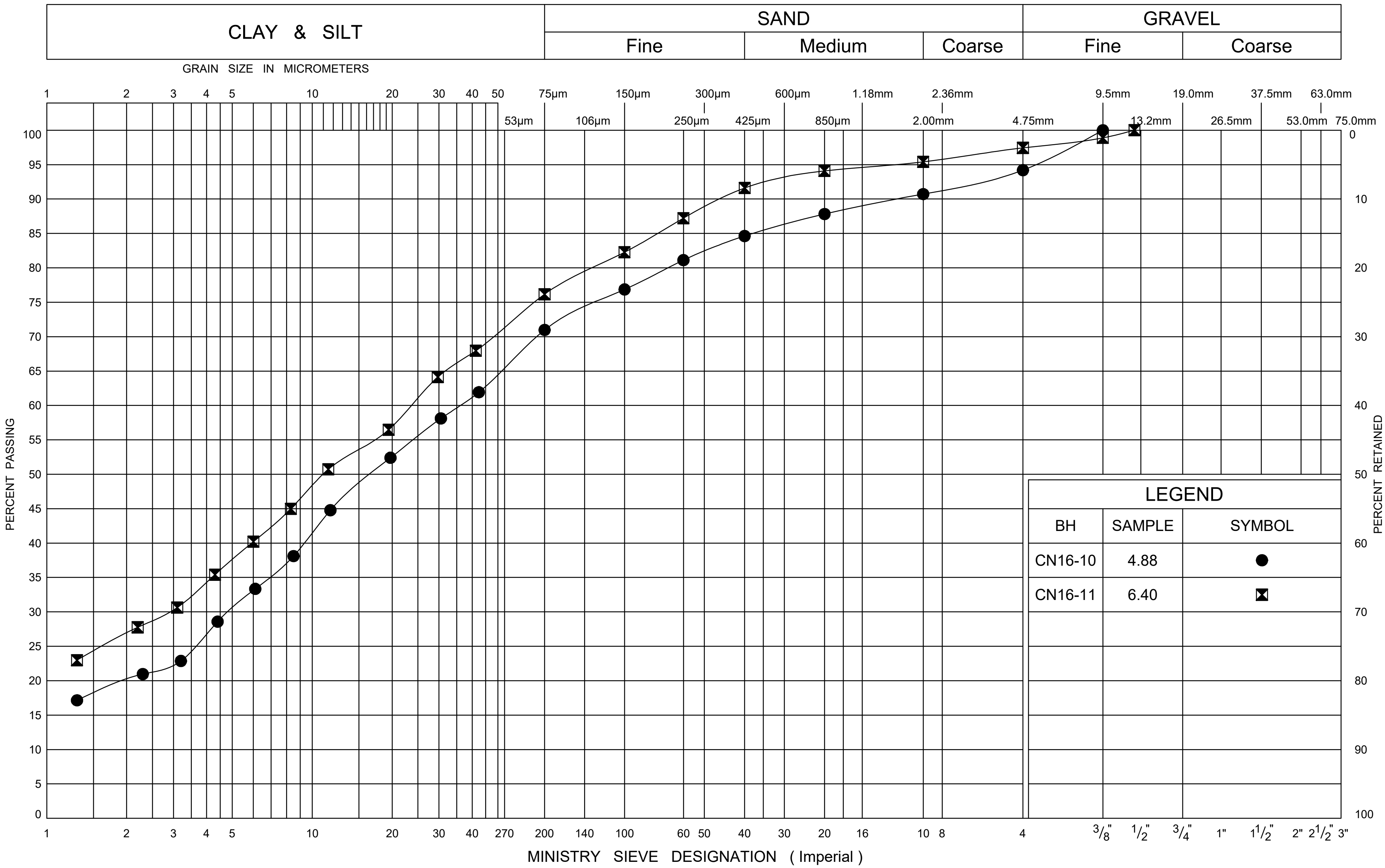
GRAIN SIZE DISTRIBUTION

SAND

FIG No A2

W P 408-88-00

78 12 M



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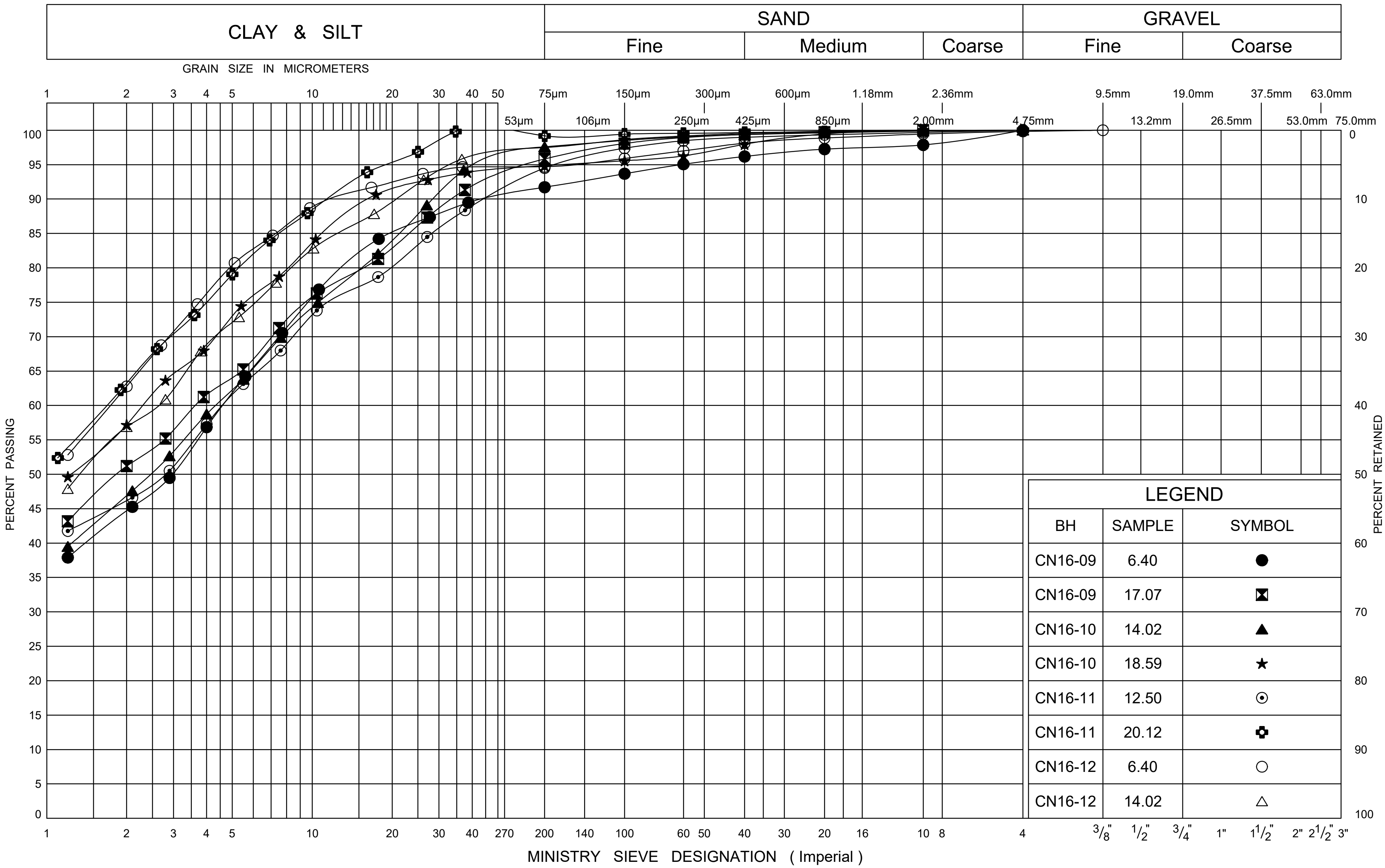
GRAIN SIZE DISTRIBUTION

Clayey SILT TILL

FIG No A3

W P 408-88-00

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
CN16-09	6.40	●
CN16-09	17.07	⧗
CN16-10	14.02	▲
CN16-10	18.59	★
CN16-11	12.50	⊙
CN16-11	20.12	✪
CN16-12	6.40	○
CN16-12	14.02	△

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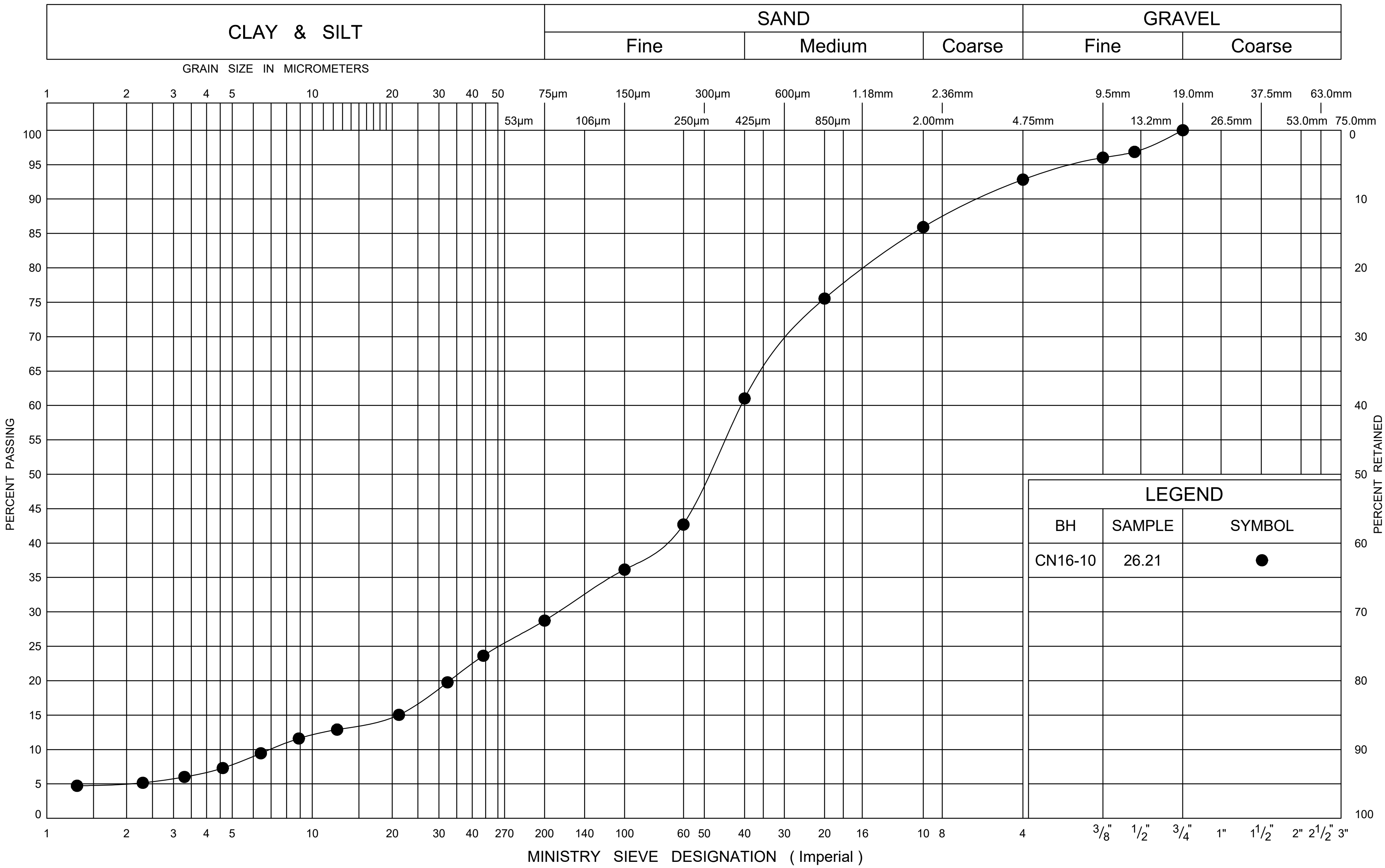
GRAIN SIZE DISTRIBUTION

Silty CLAY

FIG No A4

W P 408-88-00

78 12 M



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GRAIN SIZE DISTRIBUTION

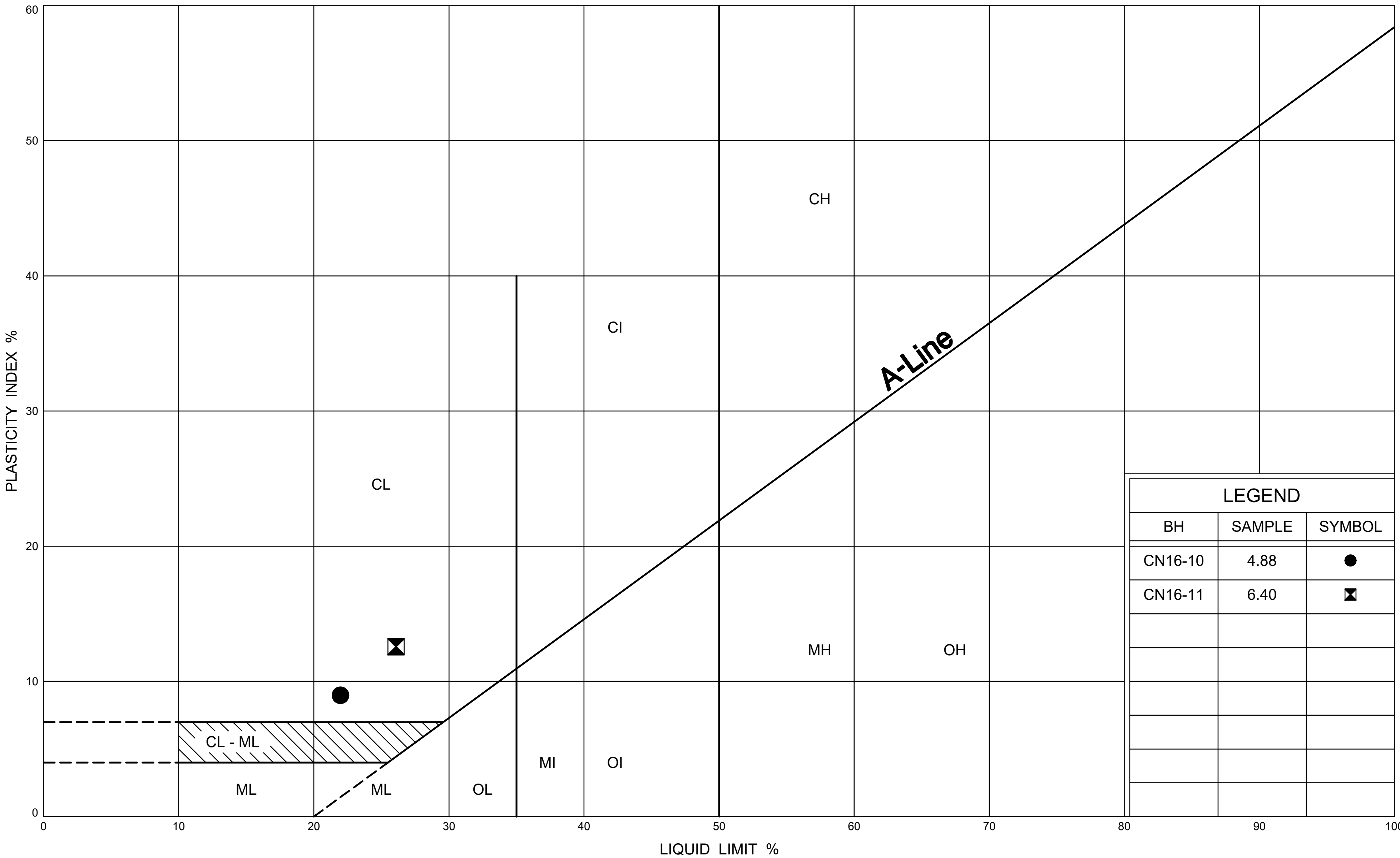
Silty SAND

FIG No A5

W P 408-88-00

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

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
CN16-10	4.88	●
CN16-11	6.40	⧗

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PLASTICITY CHART

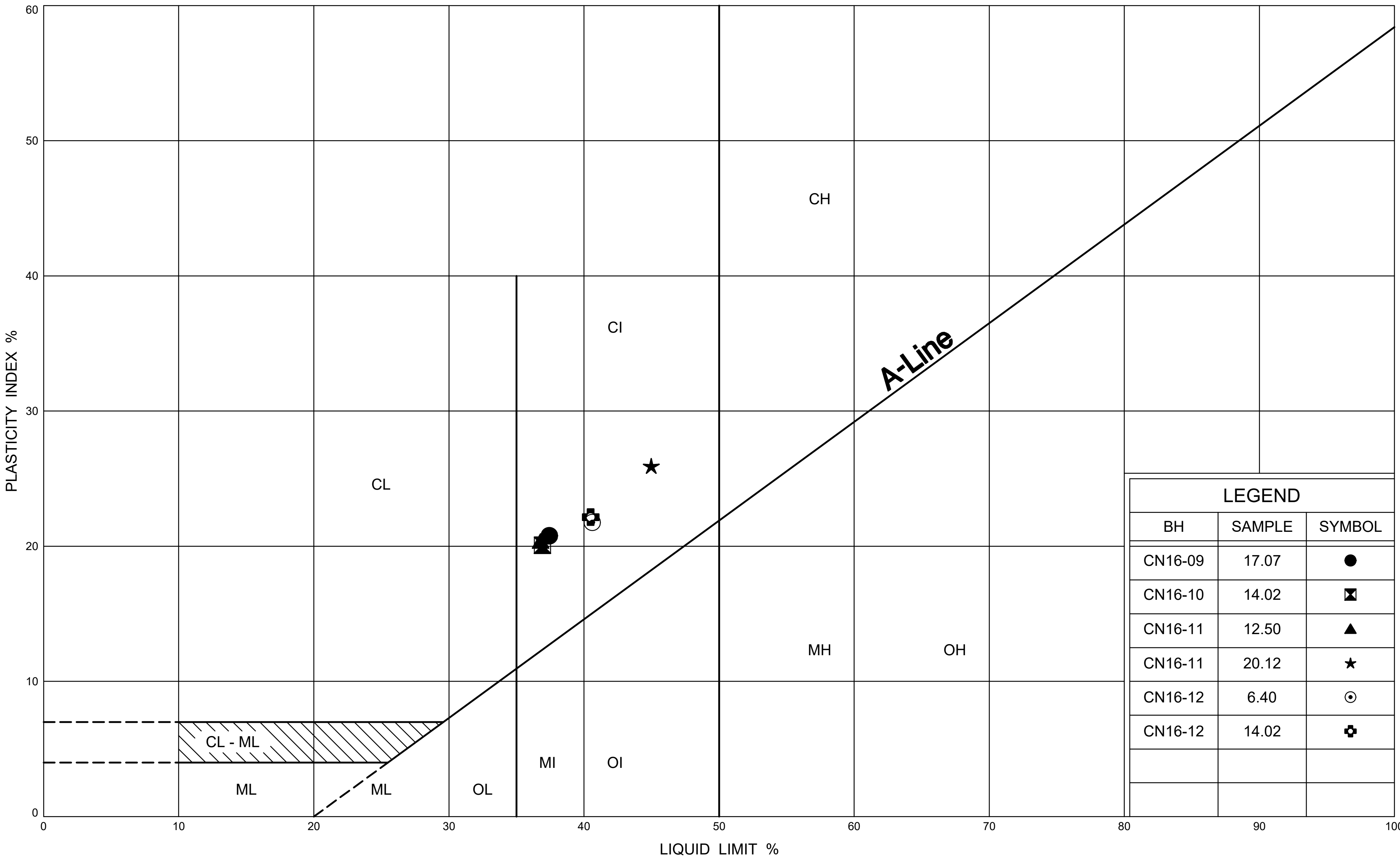
Clayey SILT TILL

FIG No A6

W P 408-88-00

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

Oct 75, FF - S - 21



LEGEND

BH	SAMPLE	SYMBOL
CN16-09	17.07	●
CN16-10	14.02	⧗
CN16-11	12.50	▲
CN16-11	20.12	★
CN16-12	6.40	⊙
CN16-12	14.02	✤

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PLASTICITY CHART

Silty CLAY

FIG No A7

W P 408-88-00

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 08-045

METRIC

G.W.P. 408-88-00 LOCATION N 4 814 152.87 E 226 282.83 ORIGINATED BY SA

HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN

DATUM Geodetic DATE 2008.08.15 - 2008.08.19 CHECKED BY RPR

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
322.2						
0.0	ASPHALT: (150mm)	1	AS			
0.2	Gravelly SAND, some silt, occasional to numerous cobbles, gasoline odour Compact to Very Dense Dark Brown to Black Moist (FILL)				322	
		1	SS	20	321	
		2	SS	67		
319.9 / 2.3	Sandy SILT, some clay, trace gravel, decayed wood fragments, mortar, glass pieces, strong gasoline odour, possible contamination, occasional organics Very Loose to Loose Black Wet (FILL)				320	
		3	SS	7		
		4	SS	1	319	
317.7 / 4.6	SAND, some gravel, some silt, trace clay Compact Grey Wet				318	
		5	SS	25	317	16 72 12 (SI+CL)
315.8 / 6.4	Clayey SILT, some sand, trace gravel Very Stiff to Hard Grey (TILL)	6	SS	16	316	
		7	SS	35	315 / 314	0 17 55 27
	occasional silty sand seams	8	SS	26	313	

Water content (%) plotted values: 114 at sample 3.

Continued Next Page

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

ONTMT4S 6417R.GPJ 3/12/09

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 08-045

METRIC

G.W.P. 408-88-00 LOCATION N 4 814 152.87 E 226 282.83 ORIGINATED BY SA

HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN

DATUM Geodetic DATE 2008.08.15 - 2008.08.19 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						
311.6	Clayey SILT, some sand, trace gravel Very Stiff to Hard Grey (TILL)				312		
10.7	Silty CLAY, trace sand, occasional cobbles Very Stiff to Hard Grey	9	SS	38	311		
		10	SS	26	310		0 2 33 65
		11	SS	37	309, 308		
		12	SS	78	307		
	trace gravel	13	SS	101	306, 305		
		14	SS	95	304, 303		0 5 45 50

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

+ 3, × 3: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 08-045

METRIC

G.W.P. 408-88-00 | LOCATION N 4 814 152.87 E 226 282.83 | ORIGINATED BY SA

HWY 7 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY AN

DATUM Geodetic | DATE 2008.08.15 - 2008.08.19 | CHECKED BY RPR

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty CLAY, trace gravel, occasional cobbles Hard Grey	15	SS	86	302		
		16	SS	96	301		
		17	SS	59	300 / 299		
		18	SS	50	298		0 0 21 79
		19	SS	51	297 / 296		
	occasional thin grey silt layers	20	SS	57	295 / 294		
293.1 29.1	Silty SAND Very Dense Grey Wet (TILL)	21	SS	100/.275	293		

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

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 08-045

METRIC

G.W.P. 408-88-00 LOCATION N 4 814 152.87 E 226 282.83 ORIGINATED BY SA
HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2008.08.15 - 2008.08.19 CHECKED BY RPR

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
292.0 30.2	SILT, some sand, trace clay Very Dense Grey Wet (TILL) Layer of gravel (400mm)		22	SS	100/ .250		292	0 11 81 8
							291	
	Layer of gravel (200mm)							
290.1 32.1	END OF BOREHOLE AT 32.1m. BOREHOLE BACKFILLED WITH GROUT TO SURFACE.		23	SS	100/ .075			

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

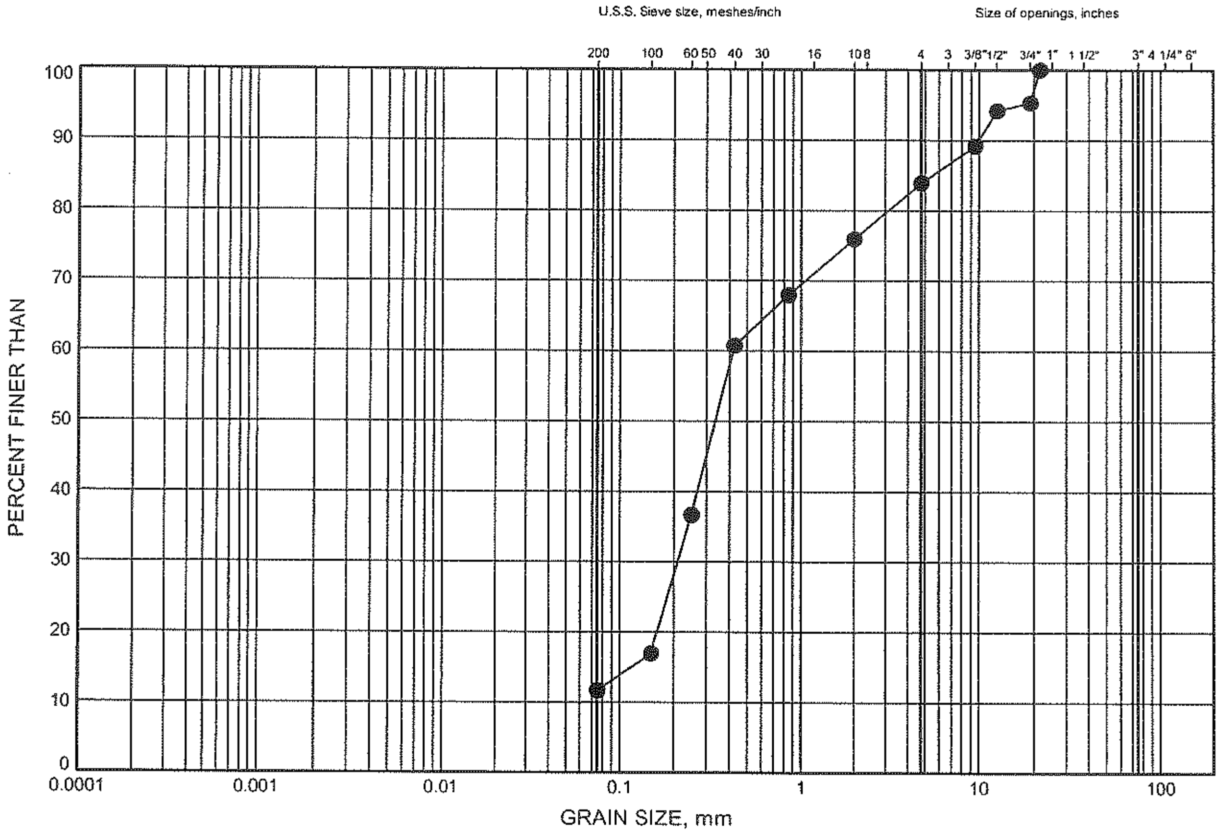
20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Highway 7 - New

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-045	4.88	317.35

W.P.# 408-88-00

Prepared By AN

Checked By RPR

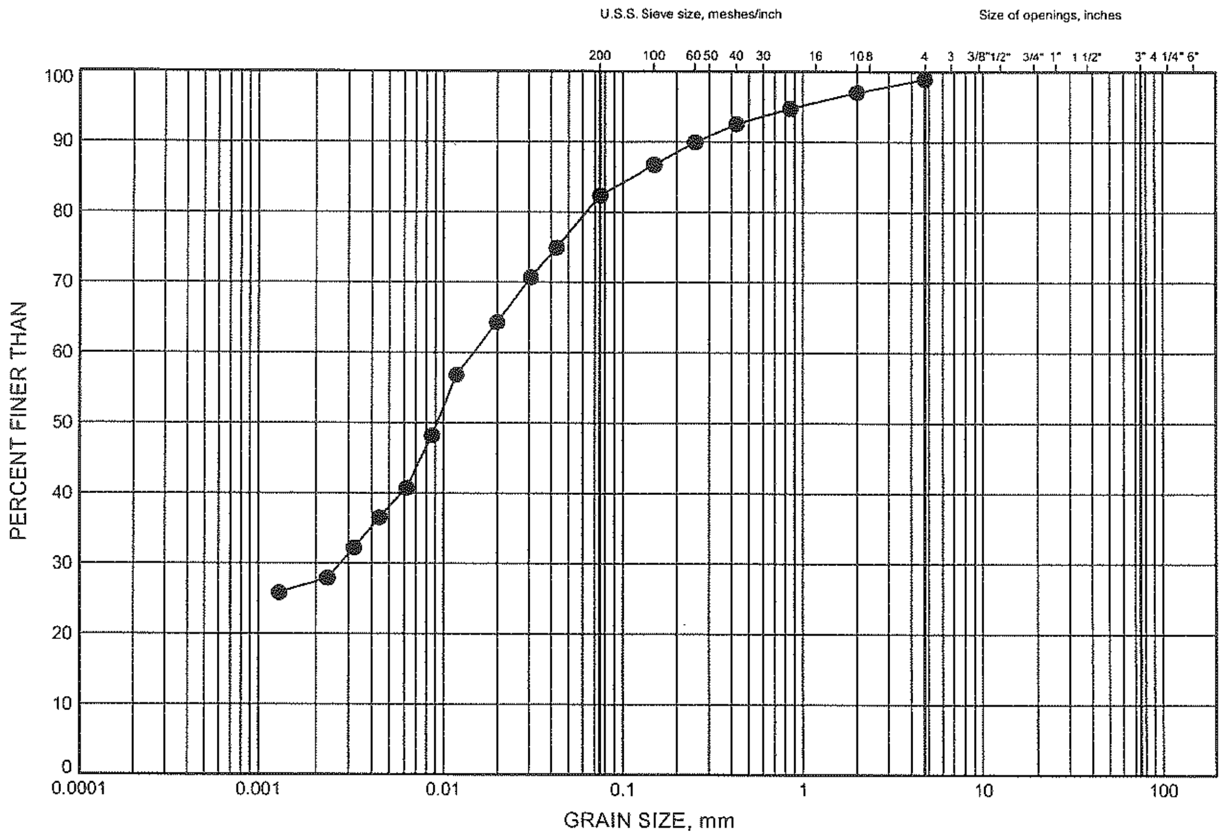
THURBER

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

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)
●	08-045	7.92	314.30

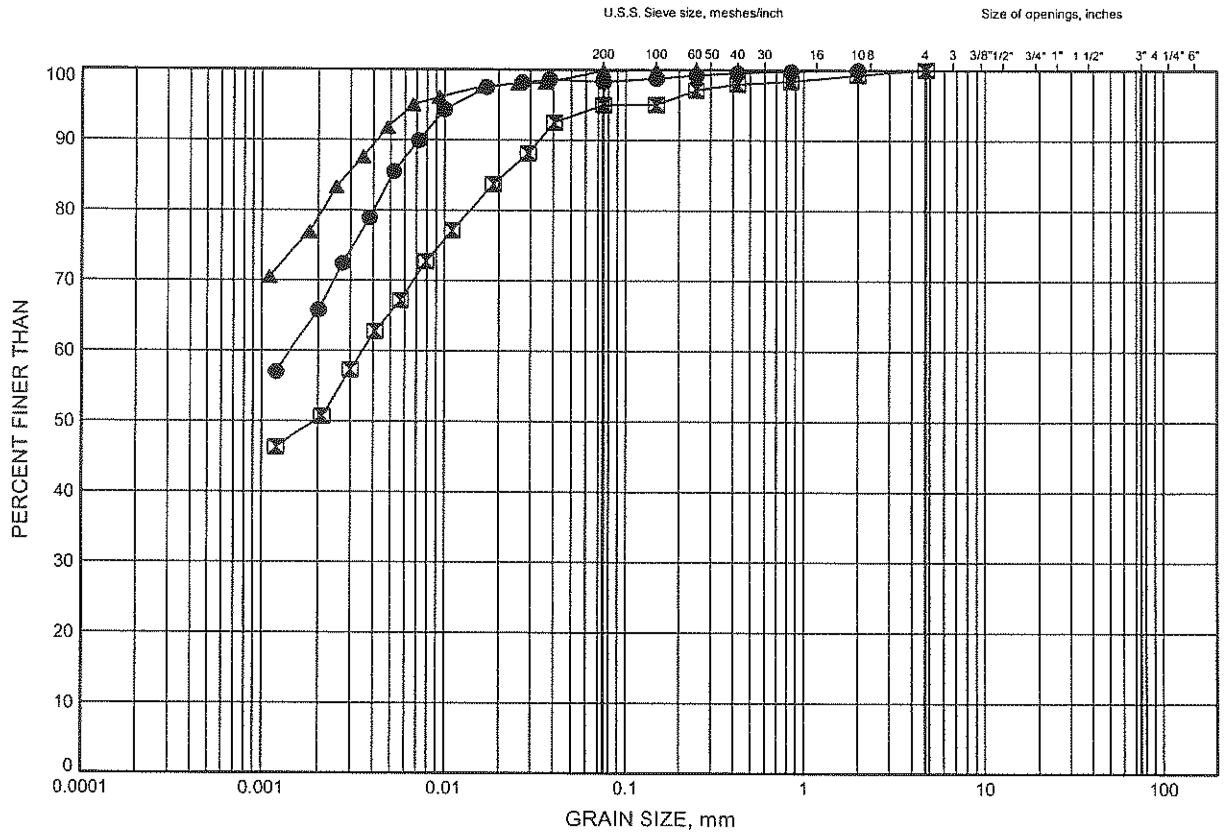
W.P.# 408-88-00
Prepared By AN
Checked By RPR

THURBER

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

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)
●	08-045	12.50	309.73
⊠	08-045	18.59	303.63
▲	08-045	24.69	297.54

W.P.# 408-88-00

Prepared By AN

Checked By RPR

THURBER

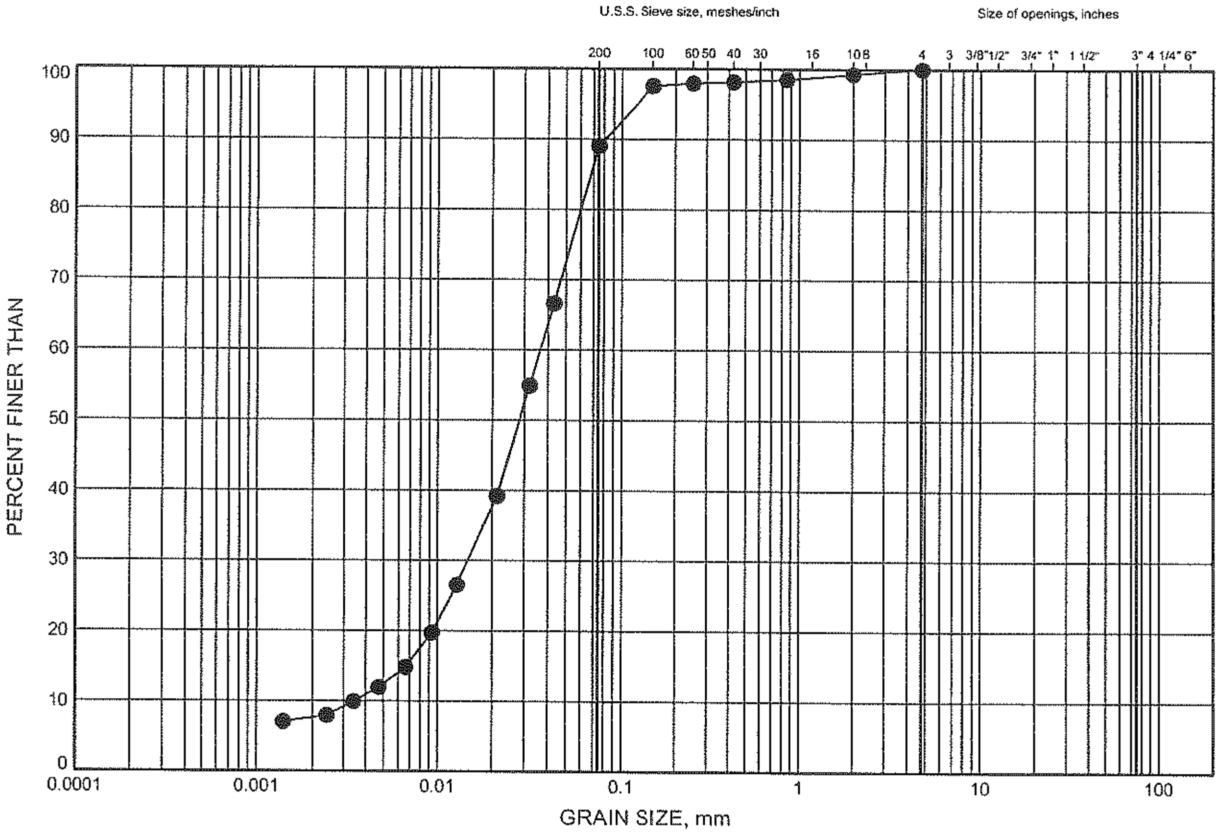
GRAIN SIZE DISTRIBUTION - THURBER 6417R.GPJ 11/24/08

Highway 7 - New

GRAIN SIZE DISTRIBUTION

FIGURE B4

SILT TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-045	30.66	291.57

W.P.# 408-88-00

Prepared By AN

Checked By RPR

THURBER

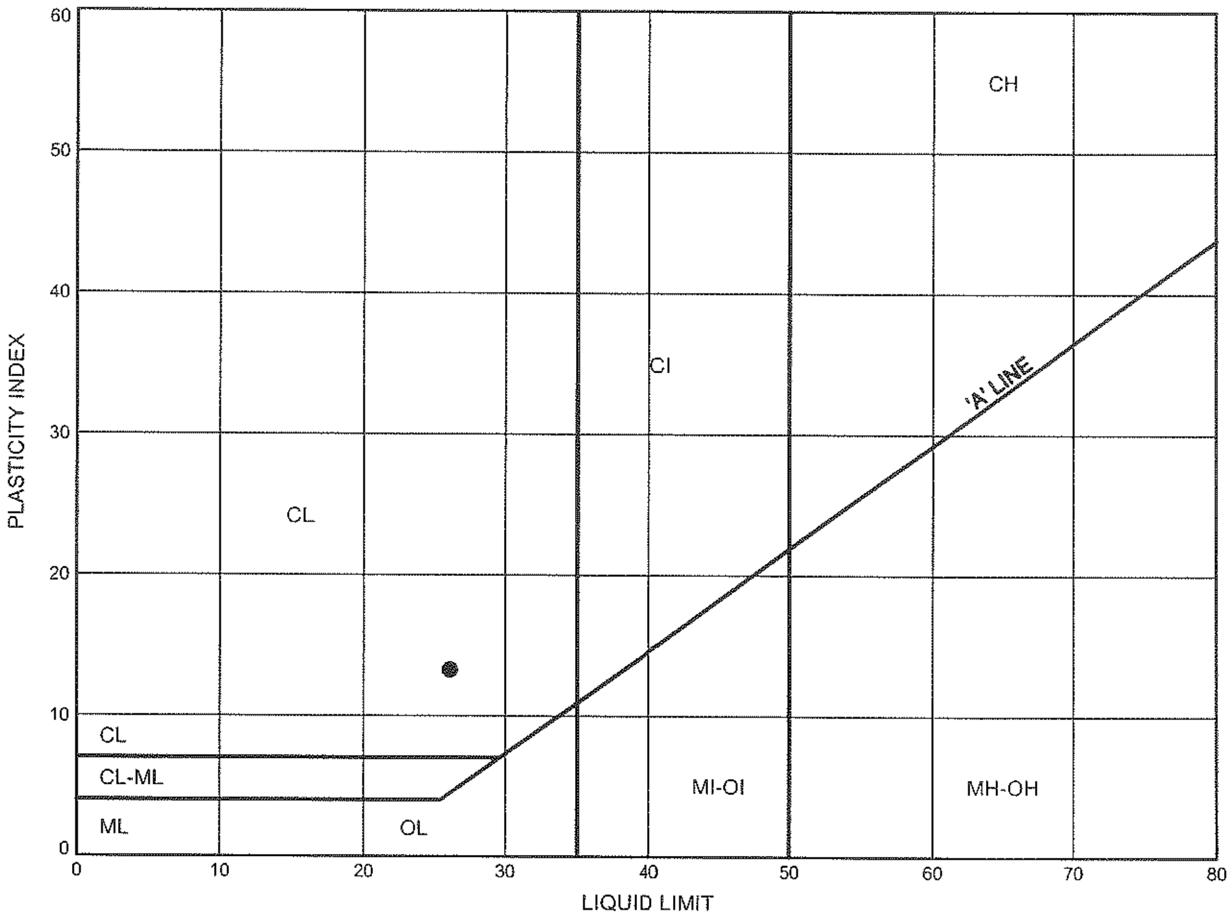
GRAIN SIZE DISTRIBUTION - THURBER 6417R.GPJ 11/24/08

Highway 7 - New

ATTERBERG LIMITS TEST RESULTS

FIGURE B5

CLAYEY SILT TILL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-045	7.92	314.30

Date November 2008

Project 408-88-00

THURBER

Prep'd AN

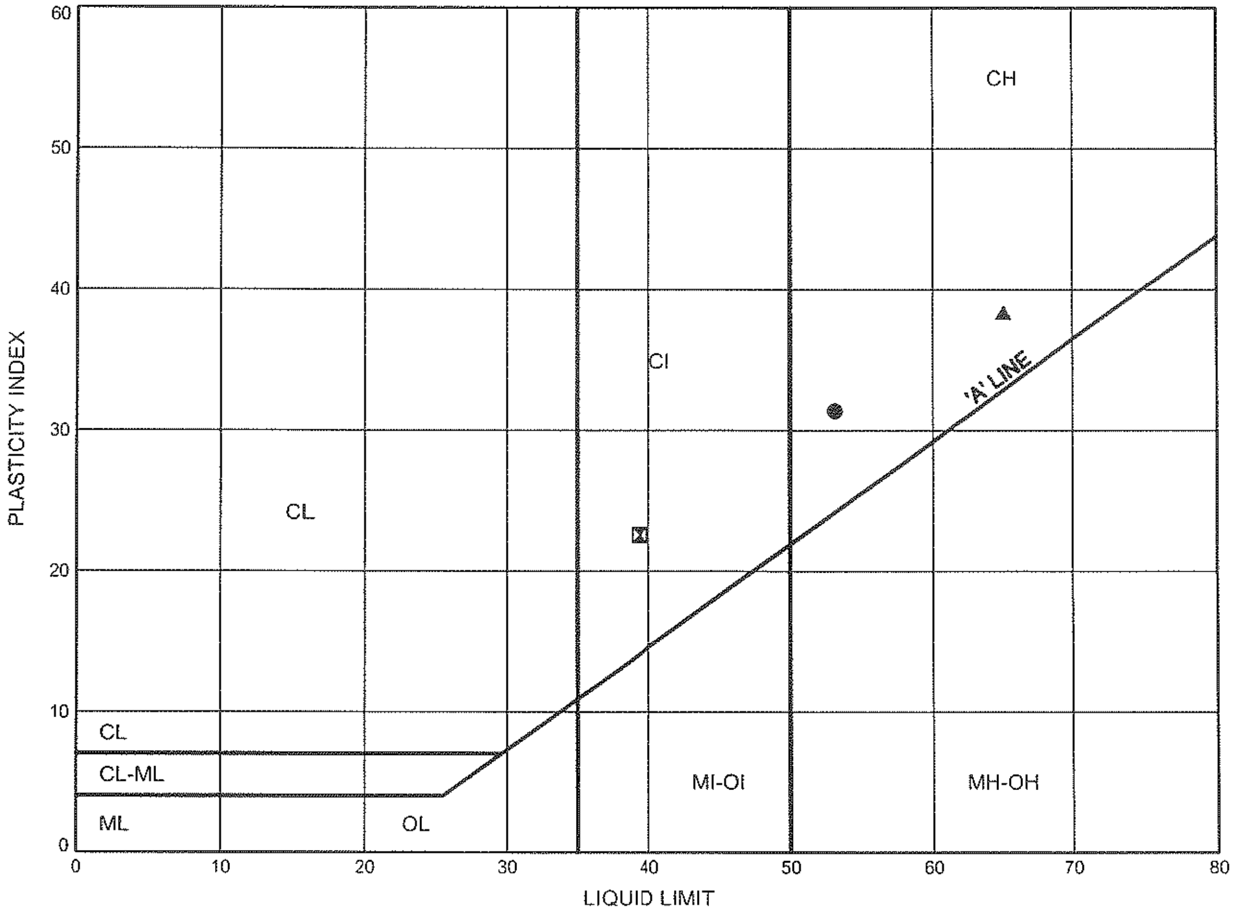
Chkd. RPR

Highway 7 - New

ATTERBERG LIMITS TEST RESULTS

FIGURE B6

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-045	12.50	309.73
⊠	08-045	18.59	303.63
▲	08-045	24.69	297.54

Date November 2008

Project 408-88-00

THURBER

Prep'd AN

Chkd. RPR

THURBALT 6417R.GPJ 11/24/08

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

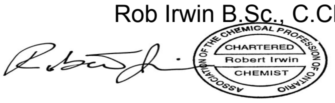


TABLE OF CONTENTS

Client: Thurber Engineering Ltd.

Project: 11375 Hwy 7 New, Kitchener

Project Manager: Nancy Berg

Samplers: Nancy Berg

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

Parameter	Units	RL		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
				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)

Parameter	Units	RL		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
				Result	Result	Result	Result	Result
General Chemistry								
Conductivity	uS/cm	2		195	317	400	1280	736

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

Parameter	Units	RL		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
				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

SGS

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		

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 COUSTODY

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

No: 009972

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): [signature]
Custody Seal Present: ☐ No
Custody Seal Intact: ☐ no

Cooling Agent Present: ☑ ice
Temperature Upon Receipt (°C): 9,9,9
LAB LIMS #: CA14437-Aug19

REPORT INFORMATION

Company: Thurber Engineering Ltd
Contact: Nancy Berg
Address: 103 - 2010 Winston Park Dr
Oakville On L6H 5R7
Phone: 647-633-8417
Email: nberg@thurber.ca
Email:

INVOICE INFORMATION

☐ (same as Report Information)
Company:
Contact:
Address:
Phone:
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 ☐ CCME ☐ MISA ☐ MMER ☐ Other:

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: [signature] — Date: 08/12/19 (mm/dd/yy) — Pink Copy - Client

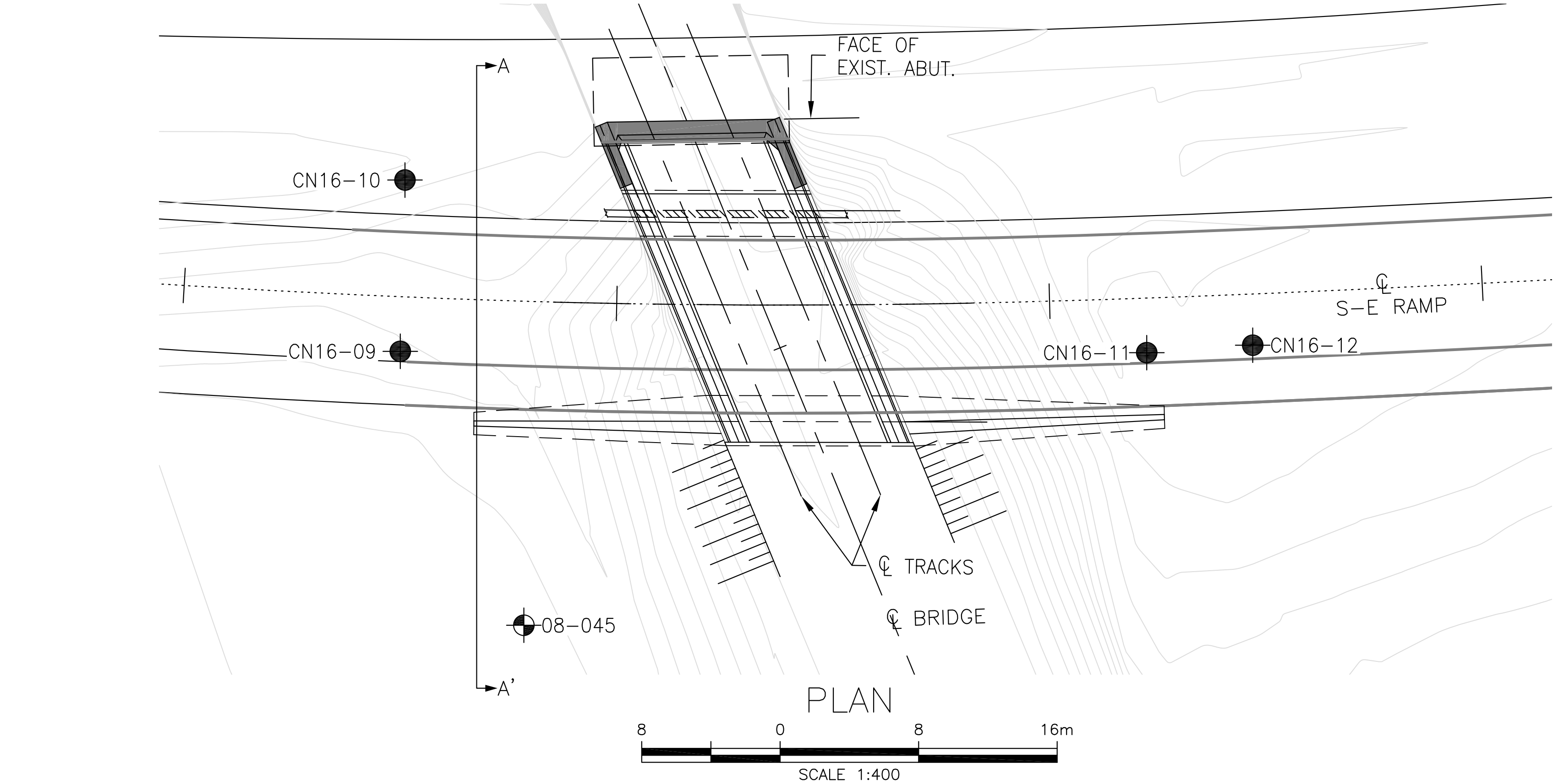
Relinquished by (NAME): Nancy Berg — Signature: [signature] — Date: 08/13/19 (mm/dd/yy) — Yellow & White Copy - SGS

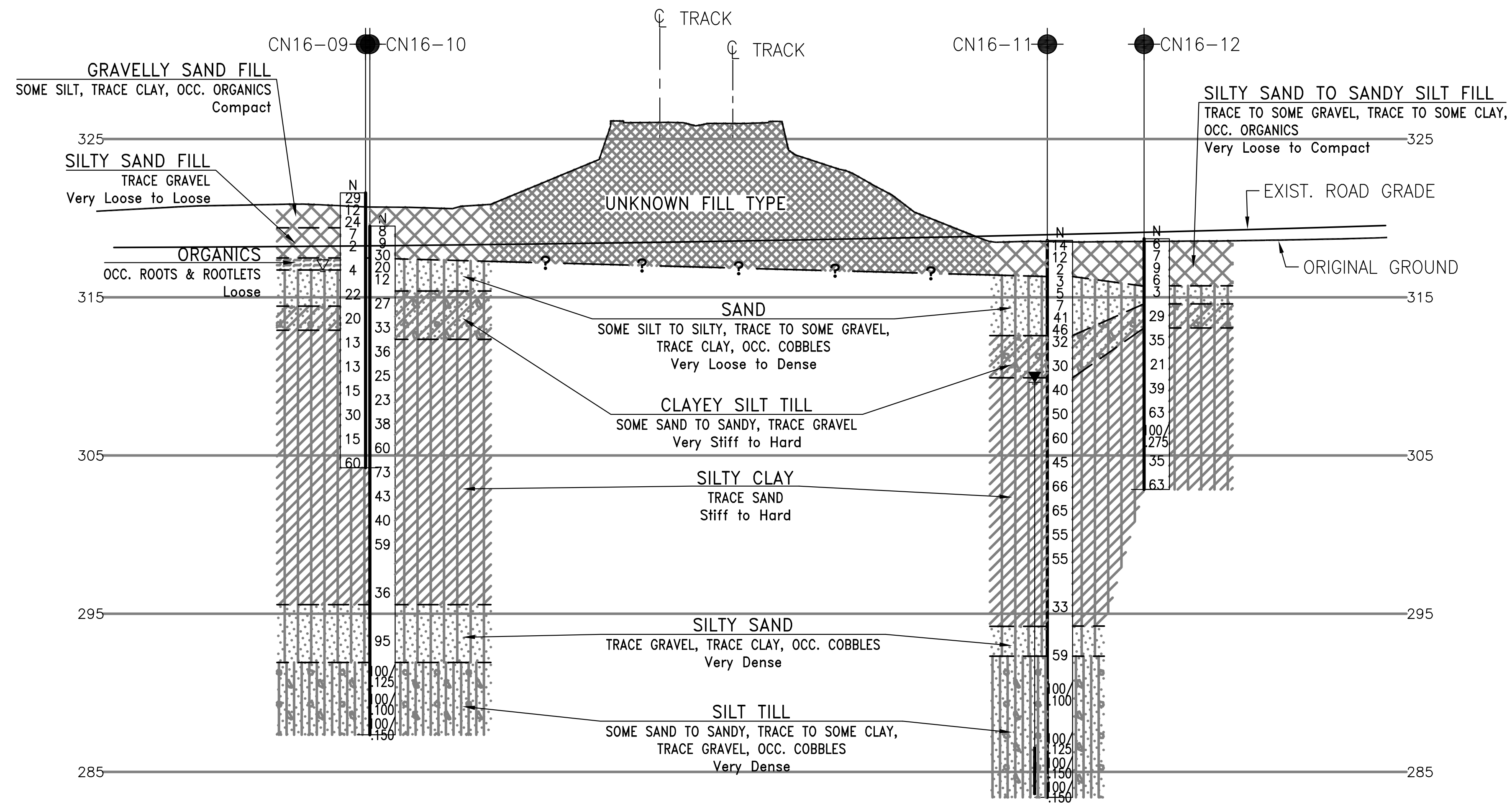
Revision #: 1.1
Date of Issue: 04 April, 2018

THURBER

Appendix D

Borehole Locations and Soil Strata Drawing





PROFILE ALONG S–E RAMP

8 0 8 16m

SCALE 1:400

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

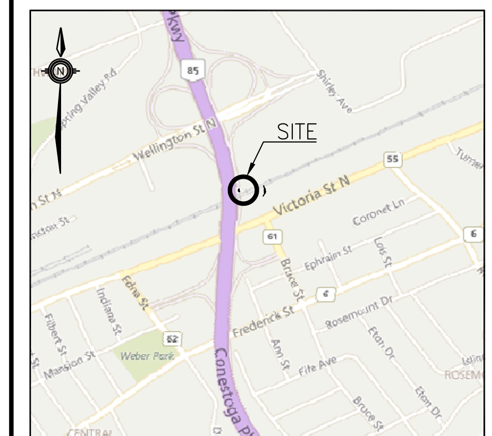
CONT No
GWP No 408–88–00

SHEET

HIGHWAY 7
S–E RAMP CONNECTION BRIDGE
UNDER METROLINX TRACKS
BOREHOLE LOCATIONS AND SOIL STRATA

WSP

THURBER ENGINEERING LTD.



Latitude: 43.457463° Longitude: –80.470548°

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
08–045	322.2	4 814 152.9	226 282.8
CN16–09	321.6	4 814 145.3	226 267.2
CN16–10	319.5	4 814 145.3	226 257.3
CN16–11	318.6	4 814 188.4	226 266.1
CN16–12	318.7	4 814 194.5	226 265.5

-NOTES-

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

GEOCRES No. 40P8-279

REVISIONS | DATE | BY | DESCRIPTION

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

FILENAME: H:\Drafting\11000\11375\TED-11375-BHPP-SE-Ramp.dwg
PLOTDATE: 7/20/2020 11:38 AM

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No

GWP No 408-88-00

HIGHWAY 7
S-E RAMP CONNECTION BRIDGE
UNDER METROLINX TRACKS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

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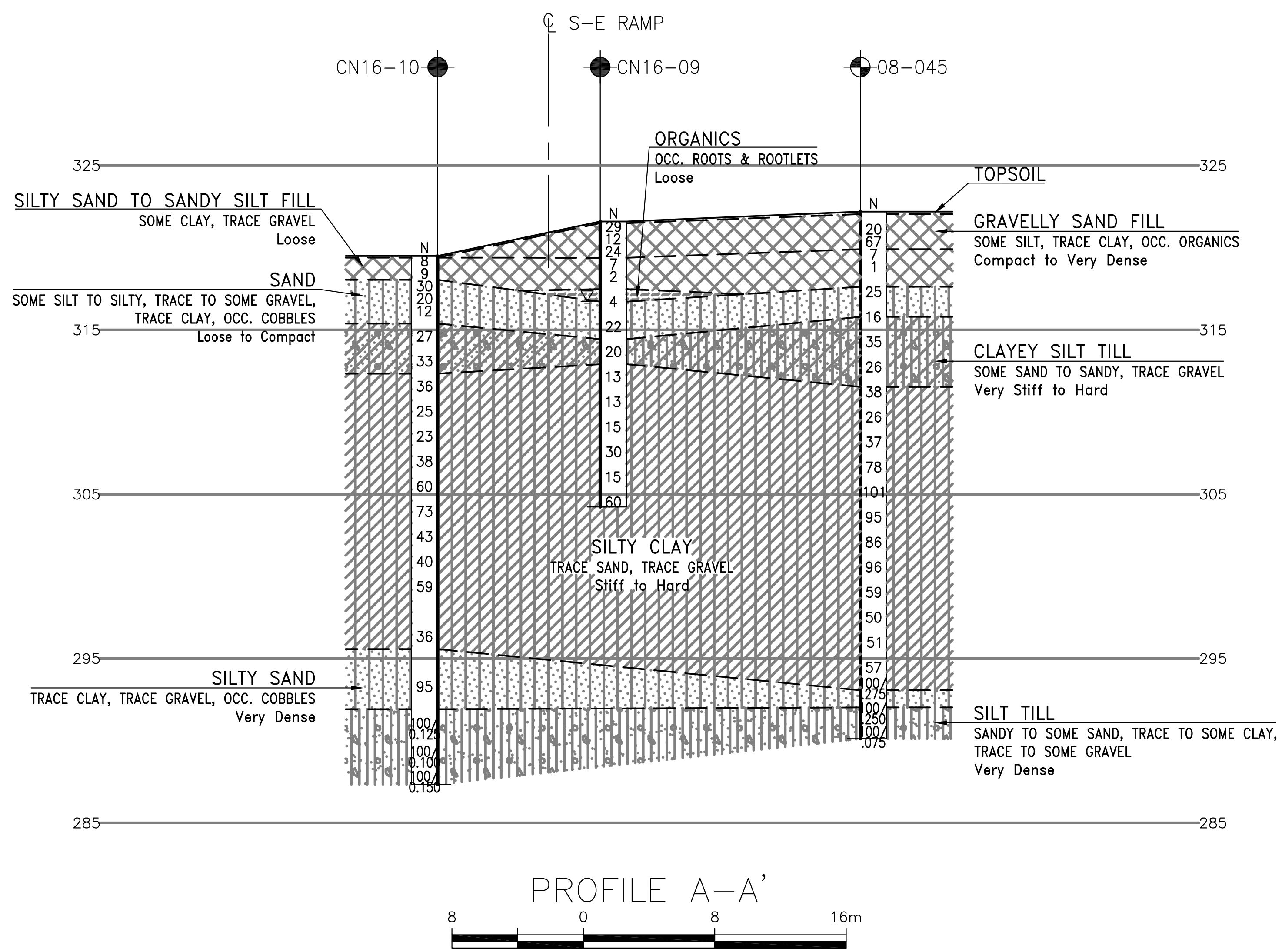
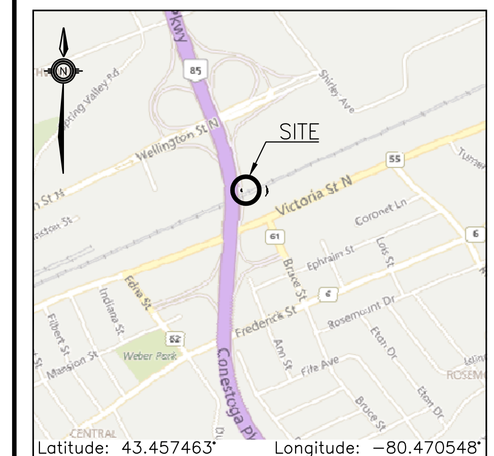


PROFILE A-A'



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
08-045	322.2	4 814 152.9	226 282.8
CN16-09	321.6	4 814 145.3	226 267.2
CN16-10	319.5	4 814 145.3	226 257.3
CN16-11	318.6	4 814 188.4	226 266.1
CN16-12	318.7	4 814 194.5	226 265.5

-NOTES-

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

GEOCRES No. 40P8-279

REVISIONS	DATE	BY	DESCRIPTION

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

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Appendix E

Site Photographs

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Photo 1: Borehole CN 16-09, looking North at the existing East Abutment of the Metrolinx Bridge

Photo 2: Borehole CN 16-10, looking North at the existing East Abutment of the Metrolinx Bridge

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Photo 3**:** Borehole CN 16-11, looking South at the existing East Abutment of the Metrolinx Bridge

Photo 4: Borehole CN 16-12, looking South at the existing East Abutment of the Metrolinx Bridge

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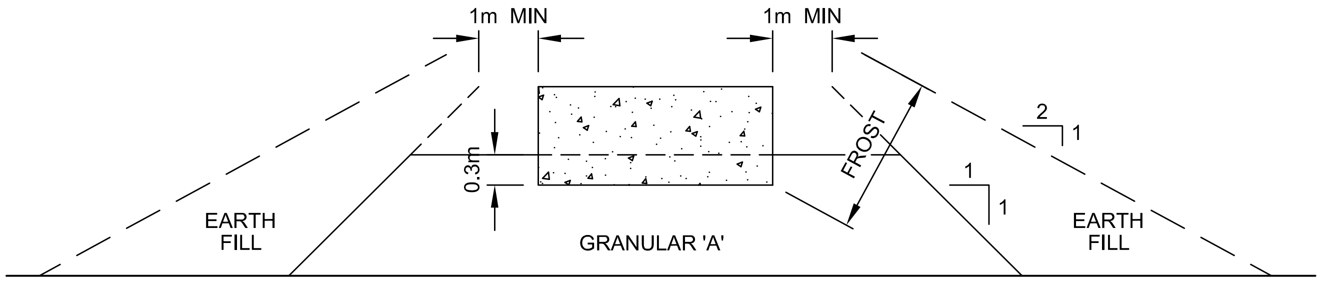
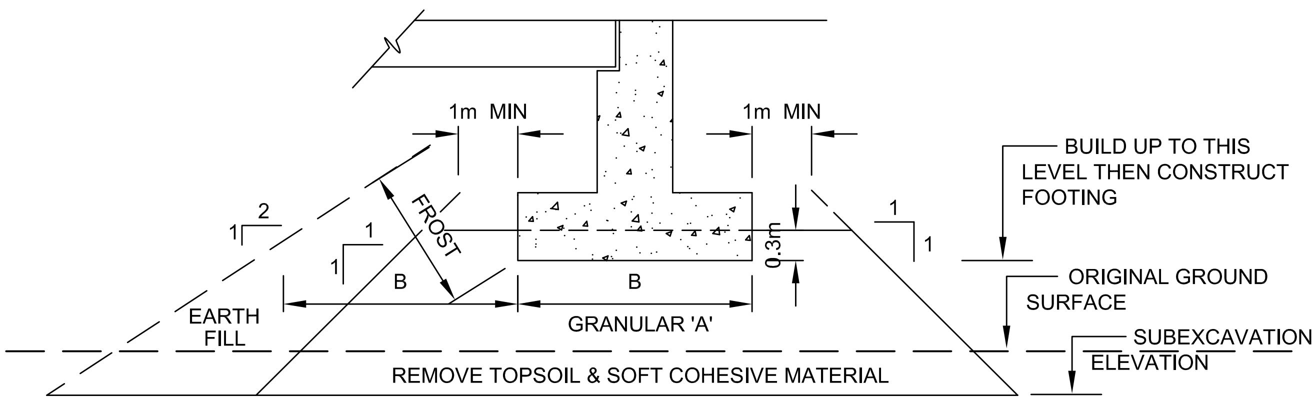
Appendix F

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

Appendix G

Foundation Comparison

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. Sub excavation 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. Installation 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. Sub excavation 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**

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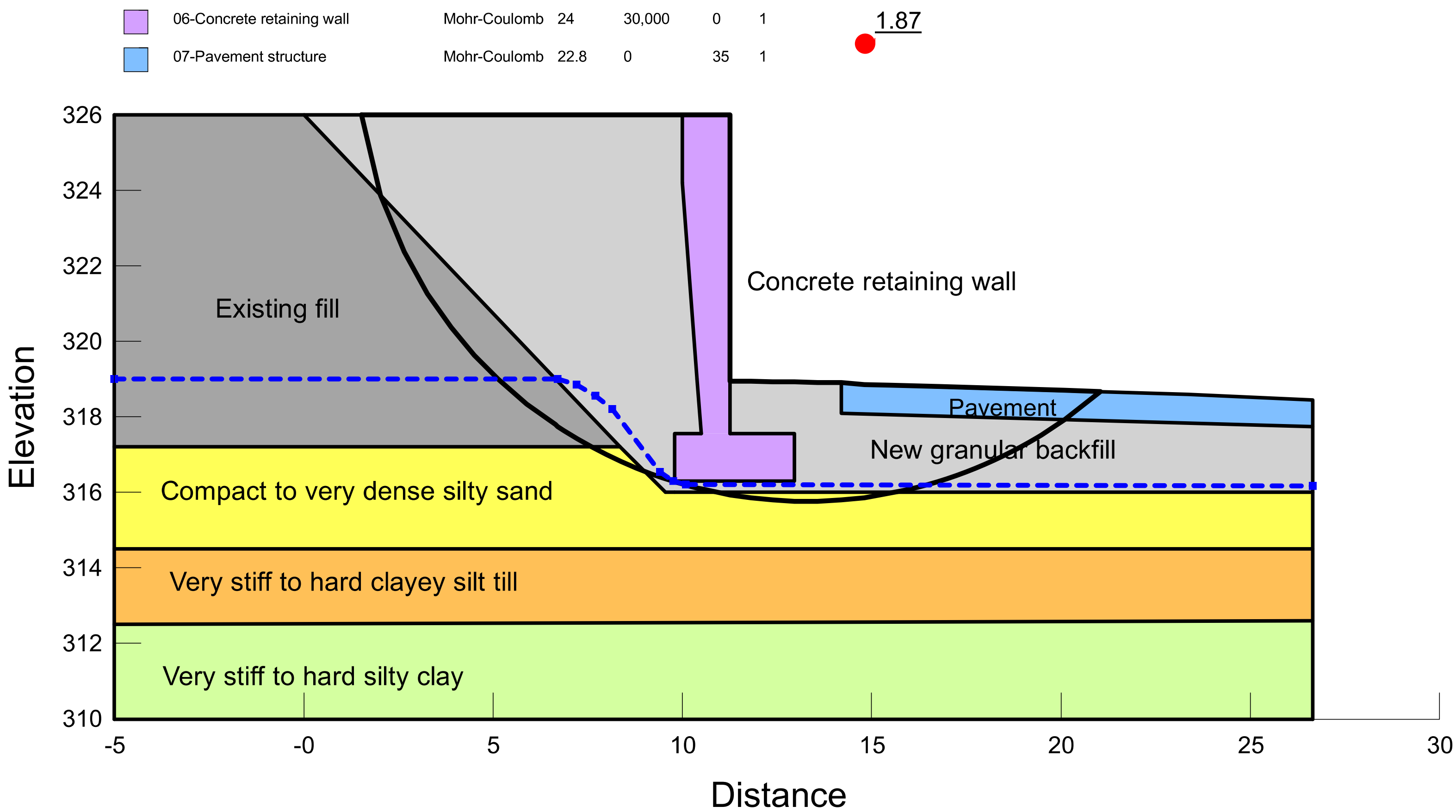
Appendix H

Slope Stability Output

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



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Project		
11375 - Hwy 7-New SE Ramp Under Metrolinx Railway		
Analysis		
Concrete Retaining Wall- Drained Analysis		
Seismic Coefficient: H: 0g, V: 0g	Last Run: 2020-05-14,07:41:31 PM	Scale: 1:185

Additional Details

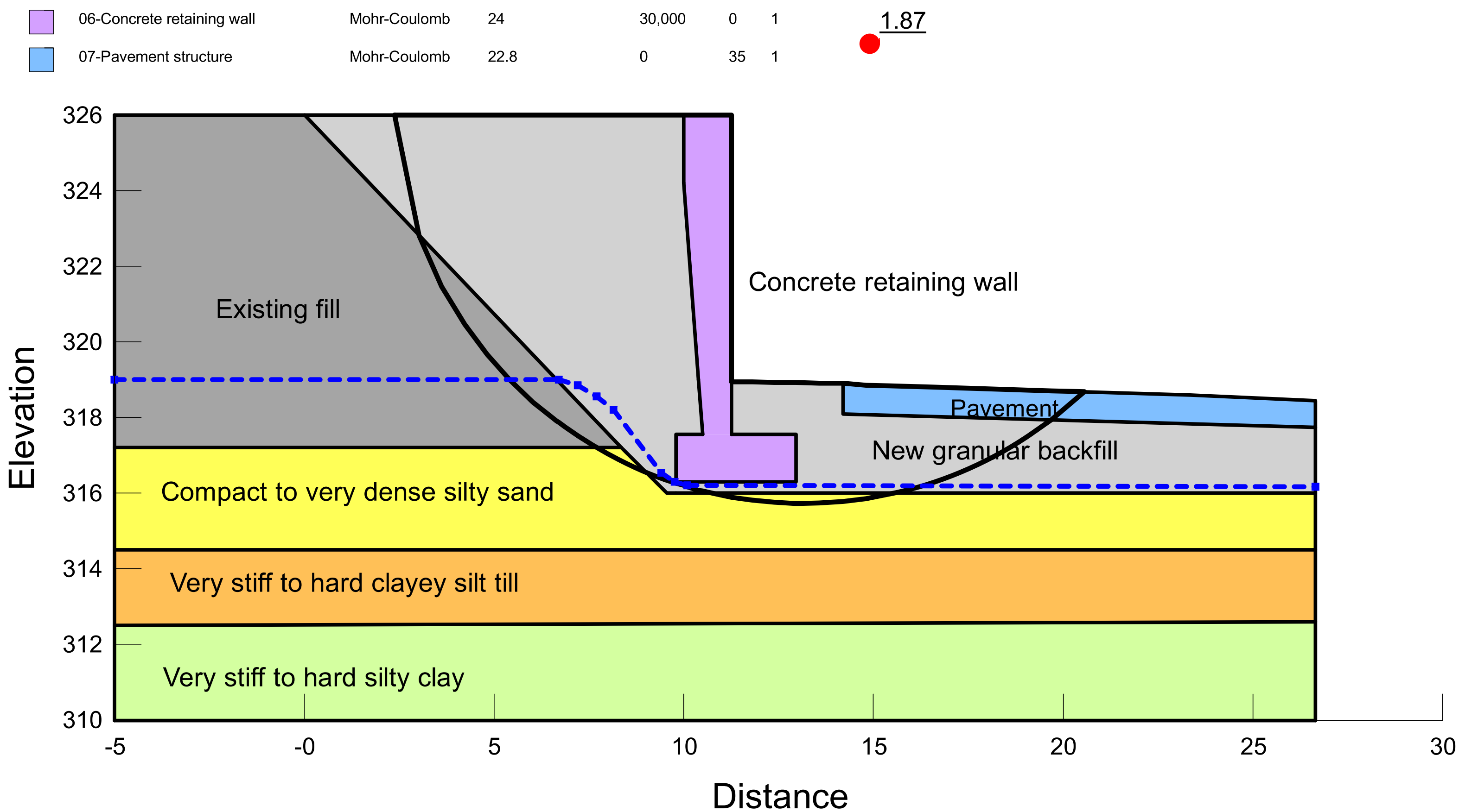
Method: Morgenstern-Price, Half-Sine

Figure H1

H:\10000+\11375 Hwy 7 New PD and DD Foundations\Reports & Memos\Rail Bridges\SE\Analysis\Slope stability\11375-CN SE Ramp- Ret Wall- drained-G1.gsz

Tool Version: 10.2.1.19666

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



Project
11375 - Hwy 7-New SE Ramp Under Metrolinx Railway

Analysis
Concrete Retaining Wall- Undrained Analysis

Seismic Coefficient
H: 0g, V: 0g

Last Run
2020-05-14,07:42:47 PM

Scale
1:185

Additional Details
Method: Morgenstern-Price, Half-Sine

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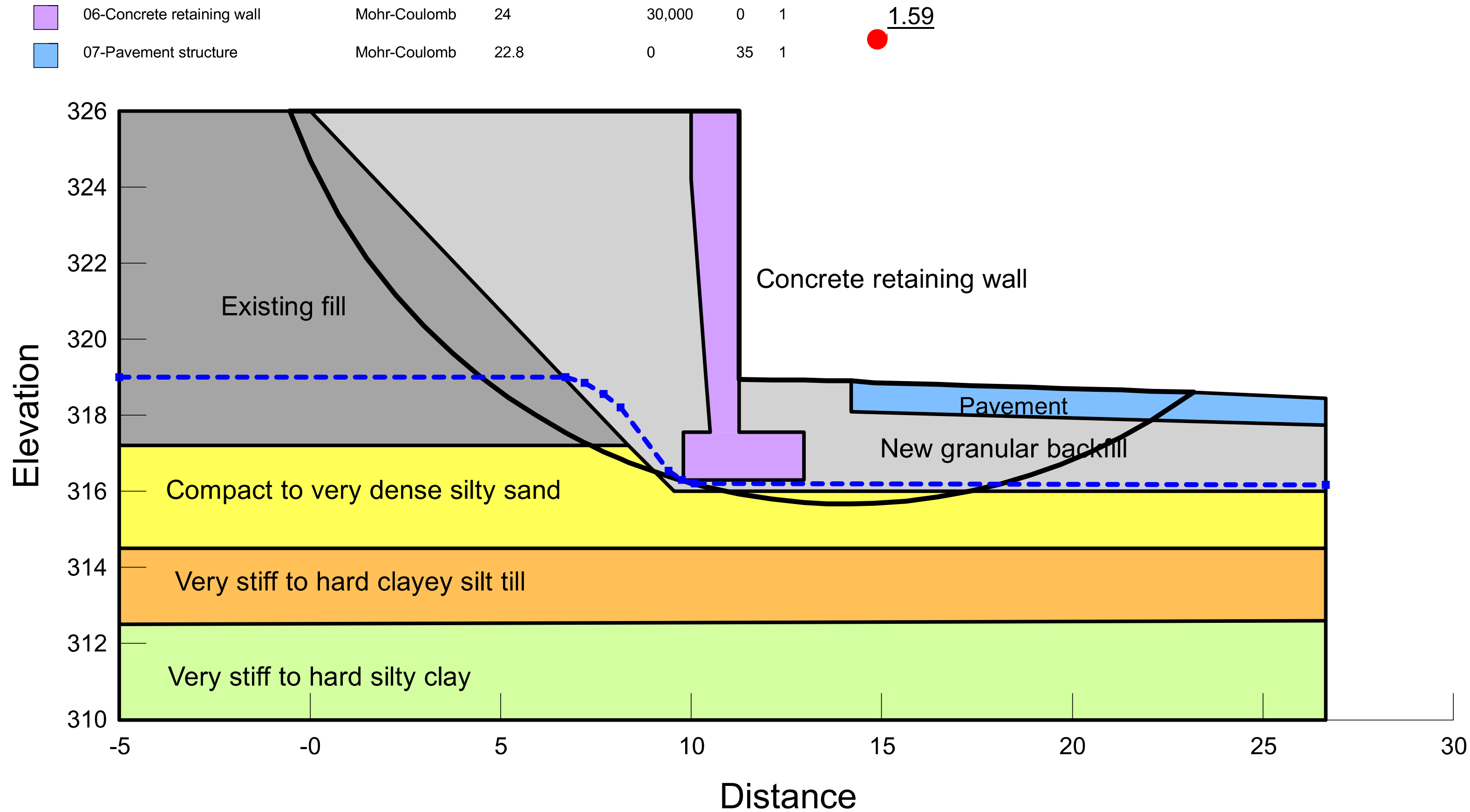
Figure H2

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



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Project
11375 - Hwy 7-New SE Ramp Under Metrolinx Railway

Analysis
Concrete Retaining Wall- Seismic Analysis

Seismic Coefficient	Last Run	Scale
H: 0.097g, V: 0g	2020-05-14,07:44:49 PM	1:185

Additional Details

Method: Morgenstern-Price, Half-Sine

Figure H3

H:\10000+\11375 Hwy 7 New PD and DD Foundations\Reports & Memos\Rail Bridges\SE\Analysis\Slope stability\11375-CN SE Ramp- Ret Wall-seismic-H3.gsz

Tool Version: 10.2.1.19666

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Appendix I

List of OPSS Documents and NSSP Wording

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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

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

Installation of H-piles shall be in accordance with OPSS.PROV 903 and the following.

The native soils at the Metrolinx bridge over the planned S-E Ramp are comprised of glacial till and are known to contain cobbles and boulders. Appropriate equipment and construction procedures will be required to penetrate or remove obstructions, such as cobbles and boulders, to permit pile installation. Pile driving must be controlled according to the criteria specified for the site.

Should a pile achieve the design ultimate geotechnical resistance or refusal at a tip elevation higher than that indicated in the contract, the Contract Administrator (CA) shall be informed immediately who should consult with the design team for resolution. Over-driving must be avoided to minimize the risk of damaging the pile.

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:

- The cobbles and boulders 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:

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- The lanes of the Kitchener-Waterloo Express way and Metrolinx tracks will be open during excavation and foundation construction of the Metrolinx bridge over the planned S-E Ramp
- 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.