THURBER ENGINEERING LTD.

DRAFT
FOUNDATION INVESTIGATION AND DESIGN REPORT
N-E RAMP/OVERPASS OVER WELLINGTON STREET
HIGHWAY 7-NEW, KITCHENER TO GUELPH
G.W.P. 408-88-00

GEOCRES No.

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Report

to

WSP

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

This report presents the factual findings obtained from a detailed foundation investigation conducted at the site of a new N-E Ramp over the proposed S-W Ramp, S-W to E Wellington Ramp and Wellington Street to E-N Ramp, in the Regional Municipality of Waterloo. The proposed N-E Ramp is part of the Highway 7-New Project.

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

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

2. SITE AND PROJECT DESCRIPTION

The site lies within the Kitchener-Waterloo Expressway (KWE) and Wellington Street interchange. At this location, the new N-E Ramp will cross over the proposed S-W Ramp, S-W to E Wellington Ramp and Wellington Street to E-N Ramp. A Retained Soil System (RSS), numbered RW-08, wall is proposed on the south side of the west abutment.

The site lies within an area of industrial and commercial lands and is generally flat.

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

3. INVESTIGATION PROCEDURES

A detailed geotechnical investigation was conducted between April 12 and May 2, 2018 and consisted of drilling five boreholes (numbered NE16-13 to NE16-17) near the proposed foundation elements of the ramp structure and one borehole (numbered RW08-01) for the proposed retaining wall (RW-08) on the south side of the west abutment. Boreholes NE16-13 and NE16-17 were drilled at the west and east approaches, respectively, and were extended to 14.3 m and 11.3 m depth (Elevations 306.3 and 312.8). Boreholes NE16-14, NE16-15 and NE16-16 were drilled near the approximate locations of the west abutment, pier and east abutment, respectively. Boreholes NE16-14 to NE16-16 ranged in depth from 20.1 m to 23.2 m (Elevations 300.8 to 301.5). Borehole RW08-01 was terminated at 8.2 m depth (Elevation 314.4).

The Record of Borehole sheets for the boreholes are included in Appendix A.

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

The ground surface elevations and coordinates of the as-drilled boreholes were provided by WSP.

Prior to commencing the site investigation, utility clearances were obtained for all borehole locations. Road occupancy permit was also obtained to complete site investigation.

During the investigation, a rubber track mounted B-57 drill rig, was used in conjunction with hollow-stem augers and tricone to advance the boreholes. Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils.

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 throughout the drilling operations. Standpipe piezometers consisting of 25 mm diameter PVC pipe with a slotted screen and enclosed in filter sand, were installed in Boreholes RW08-01, NE16-15 and NE16-17 a to permit longer-term groundwater level monitoring. The borehole completion details are also shown in Table 3.1.

The completion of the boreholes and the standpipe piezometers were carried out in accordance with the requirements of O. Reg. 903 (as amended by O. Reg. 372/07).

Table 3.1 – Borehole Completion Details

Foundation Unit	Borehole	Borehole Depth / Base Elevation (m)	Piezometer Tip Elevation (m)	Completion Details
West Approach	NE16-13	14.3/306.3	None installed	Borehole backfilled with bentonite holeplug from 14.3 m to 0.3 m and auger cuttings to surface.
West Abutment	NE16-14	20.1/301.3	None Installed	Borehole backfilled with bentonite holeplug from 20.1 m to 0.3 m and auger cuttings to surface.
Pier	NE16-15	21.7/301.5	21.4/301.8	Piezometer with 3.0 m slotted screen installed with holeplug from 21.7 m to 21.4 m, sand filter from 21.4 m to 17.5 m, bentonite mixed with auger cuttings from 17.5 m to ground surface.
East Abutment	NE16-16	23.2/300.8	None Installed	Borehole backfilled with bentonite holeplug from 23.2 m to 0.3 m and cuttings to surface.
East Approach	NE16-17	11.3/312.8	11.3/312.8	Piezometer with 3.0 m slotted screen installed with sand filter from 11.3 m to 7.9 m, bentonite mixed with auger cuttings from 7.9 m to ground surface.
Retaining wall (RW-08, south side of west abutment and approach)	RW08-01	8.2/314.4	7.6/315.0	Piezometer with 3.0 m slotted screen installed with holeplug from 8.2 m to 7.7 m, sand filter from 7.7 m to 4.0 m, bentonite mixed with auger cuttings from 4.0 m to ground surface.

4. LABORATORY TESTING

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

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

5. DESCRIPTION OF SUBSURFACE CONDITIONS

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

In general, the soil stratigraphy at this site consists of surficial topsoil, granular and cohesive fill overlaying layers of native silty clay till and silty sand, which are underlain by deposits of silty clay and sand and silt till. Descriptions of the individual strata are presented below.

5.1 Topsoil

A topsoil layer ranging from 50 mm to 700 mm in thickness was encountered at the ground surface in Boreholes NE16-13, NE16-15, NE16-16 and NE16-17.

The natural moisture content ranged from 11 percent to 15 percent.

The topsoil thickness may vary between and beyond the borehole locations, and the limited data presented in this report should not be used for quantity estimation purposes.

5.2 Fill

A layer of fill was encountered below the topsoil in Boreholes NE16-13, NE16-15, NE16-16 and NE16-17 and, surficially in Boreholes NE16-14 and RW08-01. The fill consisted of cohesive and cohesionless soils.

The cohesionless fill consisted of a layer of brown sand with trace to some silt and trace to some gravel. The sand fill layer ranged in thickness from 0.4 m to 2.9 m.

A 0.7-m thick layer of brown clayey silt fill with some sand to sandy, and occasional rootlets and cobbles was contacted surficially in Borehole NE16-14.

The depth to the base of the fill ranged from 0.7 m to 3.3 m (Elevations 321.9 to 319.9).

The cohesionless fill is classified as compact, based on SPT 'N' values ranging from 10 to 29 blows for 0.3 m of penetration. An SPT 'N' value of 65 blows per 0.3 m of penetration, indicating a very dense state, was measured in Borehole NE16-17 near Elevation 322.4. An SPT 'N' value measured in the clayey silt fill was 8 blows per 0.3 m of penetration, indicating a firm consistency. The natural moisture content ranged from 3 percent to 17 percent.

Grain size distribution curves of the sand fill is presented on the Record of Borehole sheets in Appendix A, and on Figures B1 of Appendix B. The result of a laboratory test carried out on a selected sample are as follows:

Soil Particle	Sand Fill Percentage (%)
Gravel	0 to 8
Sand	80 to 92
Silt	5
Clay	3
Silt and Clay	8 to 12

5.3 Silty Clay Till

A layer of native brown to grey silty clay till containing trace sand to sandy and trace gravel was contacted below the fill in Boreholes NE16-13, NE16-14, NE16-17 and RW08-01 at depths ranging from 0.7 m to 2.2 m (Elevations 321.9 to 320.0). The thickness of the silty clay till ranged from 1.5 m to 2.7 m.

The depth to the base of the silty clay till varied from 2.2 m to 4.6 m (Elevations 319.5 to 317.3).

SPT 'N' values measured in the silty clay till varied from 12 to 36 blows per 0.3 m of penetration, indicating a stiff to hard consistency. The natural moisture content ranged from 9 percent to 14 percent.

Grain size distribution curves of the silty clay till is presented on the Record of Borehole sheets in Appendix A, and on Figure B2 of Appendix B. The result of a laboratory test carried out on a selected sample are as follows:

Soil Particle	Silty clay till Percentage (%)
Gravel	0 to 5
Sand	14 to 28
Silt	42 to 58
Clay	25 to 28

The results of Atterberg Limits are presented on the Record of Borehole sheets in Appendix A and on Figure B6 of Appendix B. The results of Atterberg Limits testing are summarized below:

Liquid Limit	21 to 24
Plastic Limit	12 to 15
Plasticity Index	9

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

Glacial tills inherently contain cobbles and boulders.

5.4 Silty Sand

Layers of native brown to grey silty sand containing trace to some clay and trace to some gravel were contacted below the silty clay till and sand fill at depths ranging from 2.2 m to 4.6 m (Elevations 321.0 to 317.3) in all the boreholes. The thickness of the silty sand layer ranged from 2.1 m to 8.7 m.

The depth to the base of the silty sand ranged from 6.2 m to 11.7 m (Elevations 315.2 to 312.3). A 0.8 m thick layer of silty sand was also encountered within the silty clay layer at a depth of 7.9 m (Elevation 312.7), in Borehole NE16-13.

Boreholes NE16-17 and RW08-01 were terminated in the silty sand layer at 11.3 m and 8.2 m depth (Elevations 312.8 and 314.4), respectively.

The SPT 'N" values of the silty sand ranged from 21 to over 101 blows per 0.3 m of penetration indicating a compact to very dense relative density, typically very dense. Higher SPT 'N' values of 100 blows per 0.125 m to 0.175 m of penetration, indicating a very dense state, were also measured in the silty sand, in Boreholes NE16-15 and NE16-16. The SPT 'N' value of 3 blows per 0.3 m of penetration measured at a depth of 7.6 m, in Borehole RW08-01, was likely due to soil disturbance. The natural moisture contents generally lay in the range of 3 percent to 21 percent.

Grain size distribution curves for the silty sand samples tested are presented on the Record of Borehole sheets in Appendix A and on Figure B3 of Appendix B. The results of gradation tests carried out on selected sampled are summarized follows:

Soil Particles	Percentage (%)
Gravel	0 to 15
Sand	47 to 70
Silt	17 to 43
Clay	6 to 11

5.5 Silty Clay

A layer of brown silty clay was encountered below the silty sand at depths ranging from 6.2 m to 11.7 m (Elevations 315.2 to 312.3) in Boreholes NE16-13 to NE16-16. The silty clay layer ranged in thickness from 7.7 m to 10.1 m.

The depth to the base of the silty clay ranged from 16.3 m to 19.4 m (Elevations 305.4 to 304.6). Borehole NE16-13 was terminated in the silty clay layer at 14.3 m (Elevation 306.3).

SPT 'N' values in the silty clay ranged from 23 to 85 blows per 0.3 m of penetration, indicating a very stiff to hard consistency. The natural moisture contents generally lay in the range of 12 percent to 28 percent.

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Grain size distribution curves for the silty clay samples tested are presented on the Record of Borehole sheets in Appendix A and on Figure B4 of Appendix B. The results of gradation tests carried out on selected sampled are summarized follows:

Soil Particles	Percentage (%)
Gravel	0
Sand	0
Silt	37 to 73
Clay	27 to 63

The results of Atterberg Limits are presented on the Record of Borehole sheets and in Figure B7 included in Appendix B. The results of Atterberg Limits testing are summarized below:

Liquid Limit	30 to 39
Plastic Limit	15 to 17
Plasticity Index	18 to 22

The above results show that the silty clay is of low to medium plasticity with group symbols of CL and CI.

5.6 Sand and Silt Till

A layer of brown sand and silt till with some clay, trace gravel and occasional cobbles was encountered below the silty clay in Boreholes NE16-14, NE16-15 and NE16-16 at depths ranging from 16.3 m to 19.4 m (Elevation 305.4 to 304.6). Boreholes NE16-14, NE16-15 and NE16-16 were terminated in the sand and silt till layer at depths ranging from 20.1 m to 23.2 m (Elevations 301.5 to 300.8).

The SPT 'N' values in the sand and silt till ranged from 100 blows per 0.3 m of penetration to 100 blows per 0.1 m of penetration, indicating a very dense relative density. The measured natural moisture content ranged from 8 percent to 25 percent.

Grain size distribution curves for the sand and silt till samples tested is presented on the Record of Borehole sheets in Appendix A and on Figure B5 of Appendix B. The results of a laboratory tests carried out on the samples are summarized as follows:

Soil Particles	(%)
Gravel	6 to 17
Sand	40 to 41
Silt	26 to 36
Clay	16 to 17

Auger grinding was noted during drilling in this deposit.

Although not specifically identified in the boreholes, glacial tills are known to contain cobbles and boulders

5.7 Groundwater Conditions

Groundwater conditions were observed during drilling operations, and groundwater levels were measured in the open boreholes upon completion of drilling. Standpipe piezometers were installed in Boreholes RW08-01, NE16-15 and NE16-17 to monitor the groundwater level at the site. The groundwater levels measured in the open boreholes and in the standpipe piezometers are summarized below.

Table 5.1 – Water Level Measurements

Foundation Unit	Borehole	Date	Water Level (m)		Remark
			Depth	Elevation	
West Approach	NE16-13	May 1, 2018	Mud was added during drilling; therefore, it was not possible to measure the water level upon completion of drilling		Open borehole
West Abutment	NE16-14	April 24, 2018			Open Borehole
Pier	NE16-15	May 2, 2018 May 4, 2018 May 16, 2018 May 31, 2018 June 25, 2018	3.8 3.8 13.6 14.0 14.1	319.4 319.4 309.6 309.2 309.1	Open borehole Piezometer Piezometer Piezometer Piezometer

East Abutment	NE16-16	April 17, 2018	Dry upon completion		Open Borehole
East Approach	NE16-17	April 12, 2018 April 27, 2018, May 16, 2018 May 31, 2018 June 25, 2018	7.0 7.2 7.2 7.0 6.7	317.1 316.9 316.9 317.1 317.4	Open Borehole Piezometer Piezometer Piezometer Piezometer
RSS Wall	RW08-01	May 1, 2018 May 16, 2018 May 31, 2018 June 25, 2018	6.1 6.1 6.1 5.8	316.5 316.5 316.5 316.8	Piezometer

The groundwater levels above are short-term readings, and seasonal fluctuations of the groundwater levels 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 sand fill from Borehole NE16-16 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 B.

Table 6.1 – Analytical Test Results

Parameter	Units (Soil)	Test Results NE16-16 SS 4 Depth 2.3 m Sand Fill
Sulphide	%	<0.02
Chloride	µg/g	12
Sulphate	µg/g	5.5
pH	No unit	9.19
Electrical Conductivity	µS/cm	76
Resistivity	Ohms.cm	13,200

Parameter	Units (Soil)	Test Results
		NE16-16 SS 4 Depth 2.3 m
		Sand Fill
Redox Potential	mV	164

7. MISCELLANEOUS

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

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

The drilling and sampling operations in the field, 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, EIT. Interpretation of the data and preparation of the report was carried out by Ms. R. Palomeque Reyna, P.Eng. and Dr. Nancy Berg, EIT.

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 N-E Ramp over the proposed S-W Ramp, S-W to E Wellington Ramp and Wellington Street to E-N Ramp (at the Wellington Street and Kitchener-Waterloo Expressway interchange) in the Regional Municipality of Waterloo, Ontario.

The General Arrangement (GA) drawing provided by WSP, dated July 2012, indicates that the new N-E ramp has two spans, each one 36.0 m in length and approximately 9.3 m in width, supported by two abutments and one pier. Each of the two abutments and the centre pier are designed to be supported by driven piles.

Based on proposed finished grade levels of Highway 7-New EBL and WBL structures and the existing ground surface near the proposed overpass abutments, the anticipated heights of the west and east approach embankments are presented in Table 8.1.

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Table 8.1 – Anticipated Approach Embankment Height

Foundation Unit	Borehole	Proposed finished grade levels of N-E Ramp (1)	Existing ground surface (2)	Approximate Approach Embankment Height (m)
West abutment	NE16-14	331.5	320.6 - 321.5	10.9 to 10.0
East abutment	NE16-16	330.3	324.0 – 324.1	6.3 to 6.2

(1) Finished grade level of N-E Ramp at the abutments, obtained from the GA drawings
(2) Ground surface elevation at the proposed abutment, obtained from boreholes

The forward and side embankment slopes are designed to be at an inclination of 2H:1V.

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 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) (2014) CSA S6-14, 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 S6-14 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, 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.

10. STRUCTURE FOUNDATIONS

The stratigraphy identified in the geotechnical investigations consisted primarily of topsoil and fill overlaying a layer of silty clay till and, compact to very dense silty sand. A layer of very stiff to hard silty clay was encountered below the silty sand. Very dense sand and silt till was encountered below the silty clay. The fill was presumably placed during construction of the existing highway. The groundwater levels measured in the piezometers ranged from 3.8 m and 14.1 m below the ground surface (Elevations 319.4 to 309.1).

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 pipes driven into the very dense/hard glacial till soils

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

10.1 Spread Footing on Native Soil

Spread footings bearing on native soil generally are a cost-effective form of foundation and are feasible at this site.

The existing fill is not considered suitable for the support of spread footings, and the spread footings should bear on native undisturbed very dense silty sand and hard 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.

Table 10.1 – Geotechnical Resistances for Spread Footings

Foundation Element	Borehole	Approximate Highest Founding Elevation (m)	Founding Stratum	Factored ULS_f (kPa)	Factored SLS_f (up to 25 mm settlement) (kPa)
West Abutment	NE16-14	320.0	Hard silty clay till	450	300
Pier	NE16-15	319.5	Very dense silty sand	600	400
East Abutment	NE16-16	320.0	Very dense silty sand	600	400

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 2014. The factored Geotechnical Resistance at SLS was assessed assuming a factor of 0.8 for typical degree of understanding of the subsurface conditions.

The bearing resistances in Table 10.1 are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be adjusted as shown in the CHBDC (2014) Clause 6.10.3 and Clause 6.10.4.

The geotechnical SLS values 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 silty clay till may be computed based on an ultimate coefficient of friction, tan δ, of 0.4 and 0.45 for the silty sand. A resistance Factor of 0.6 should be applied for cohesive soils and, 0.8 for cohesionless soils, as indicated in Table 6.2 in the CHBDC (2014).

The groundwater levels measured in the piezometers ranged from 3.8 m and 14.1 m below the ground surface (Elevations 319.4 to 309.1). If temporary excavations required to construct these footings extend below the water table, local groundwater control will be required to construct the footing in the dry and to prevent disturbance and base heave/base boiling of the footing base.

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

10.2 Spread Footing on Engineered Fill

Spread footings can also be founded on Granular "A" engineered fill pads, where this is beneficial to the overall design. These would be useful in the case of spread footings perched on a granular engineered fill pad within the approach embankment fill.

If an engineered fill pad is used, all topsoil 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 compact to very dense silty sand and hard silty clay till, and the highest permitted founding/base elevations at which engineered fill pads may be placed, are given in Table 10.2.

Table 10.2 – Highest Founding Elevations for Engineered Fill Pads

West Abutment (BH NE16-14)	Pier (BH NE16-15)	East Abutment (BH NE16-16)
320.0	319.5	321.0

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

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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 6.10.3 and Clause 6.10.4.

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 2014. The Factored Geotechnical Resistance at SLS was assessed assuming a factor of 0.8 for typical degree of understanding of the subsurface conditions.

The above founding elevations of engineered fill pad are at or above the measured groundwater levels.

If temporary excavations required to construct the engineered fill pad extend below the water table, local groundwater control 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 (2014).

The bases of the foundation excavations should be inspected by a geotechnical engineer to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to place the engineered fill. The Granular A for the engineered fill pad must be compacted to 100% Standard proctor maximum dry density (SPMDD) at optimum moisture content ±2% and placed in 150 mm lifts. The geometry of the fill pad must conform to the general requirements shown in Figure 1 in Appendix D.

10.3 Augered Caissons (Drilled Shafts)

Drilled shaft foundations founded on very dense silt and sand till were considered for the support of foundation loads at this site. However, augered caissons (drilled shafts) are not recommended for use as foundation support at this site due to high groundwater level and the presence of cohesionless soils potentially containing cobbles and boulders at the site. These conditions will cause caisson installation difficulties and therefore this option is not recommended and has not been developed further.

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 dense sand and 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 sand and silt till deposits are more prone to pile tip damage in comparison to H-piles.

The GA drawing indicates that the underside elevation of the abutment stem at the west abutment is approximately 323.5 m and at the east abutment is approximately 325.5 m, and at the pier it is at approximate elevation 320.5 m.

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 till were assessed based on the subsurface conditions encountered at the abutment and pier 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)	Pile Section HP 310 X 110		Pile Section HP 360 X 132	
				Factored ULS (kN)	Factored SLS (kN)	Factored ULS (kN)	Factored SLS_f (kN)
West Abutment	NE16-14	302.5	21.0	1,400	1,200	1,600	1,400

Pier	NE16-15	302.5	18.0	1,400	1,200	1,600	1,400
East Abutment	NE16-16	302.0	23.5	1,400	1,200	1,600	1,400

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)	Pile Section 324 mm diameter		Pile Section 356 mm diameter	
				Factored ULS (kN)	Factored SLS (kN)	Factored ULS (kN)	Factored SLS$_f$ (kN)
West Abutment	NE16-14	302.5	21.0	1,300	1,100	1,450	1,250
Pier	NE16-15	302.5	18.0	1,150	950	1,350	1,150
East Abutment	NE16-16	302.0	23.5	1,300	1,100	1,450	1,250

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 2014. The SLS values correspond to a maximum pile settlement of up to 25 mm. The Factored Geotechnical Resistance at SLS was assessed assuming a factor of 0.8 for typical degree of understanding of the subsurface conditions.

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

10.4.2 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	=	$67\ C_u / B$ (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, Sand and Silt Till (cohesionless soils)

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

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

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

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

Table 10.5 – Recommended Geotechnical Parameters for Lateral Resistance Design

Location	Reference Boreholes	Approx. Elevation (m)	Undrained Shear Strength C_u (kPa)	Unit Weight γ (kN/m^3)	K_p	n_h (kN/m^3)	Soil Conditions
West Abutment	NE16-14	321.5 to 320.0	-	20	3.0	2,500	Compact Sand, Firm Clayey Silt Fill
		320.0 to 317.3	150	20	-	-	Very Stiff to Hard Silty Clay Till
		317.3 to 315.2	-	11*	3.5	6,000	Very Dense Silty Sand
		315.2 to 305.2	200	10*	-	-	Very Stiff to Hard Silty Clay
		305.2 to 301.3	-	11*	3.5	8,000	Very Dense Sand and Silt Till
Pier	NE16-15	322.5 to 320.0	-	20	3.0	2,500	Compact Sand Fill
		320.0 to 313.7	-	11*	3.5	6,000	Compact to Very Dense Silty Sand
		313.7 to 305.5	200	10*	-	-	Hard Silty Clay
		305.5 to 302.5	-	11*	3.8	8,000	Very Dense Sand and Silt Till
East Abutment	NE16-16	324.0 to 321.0	-	20	3.0	2,500	Compact Sand Fill
		321.0 to 315.0	-	11*	3.5	6,000	Compact to Very Dense Silty Sand
		315.0 to 312.3	-	11*	3.7	7,000	Very Dense Silty Sand
		312.3 to 304.6	200	11*	-	-	Hard Silty Clay
		304.6 to 302.0	-	11*	3.8	8,000	Very Dense Sand and 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 C6.11.3(r), C6.11.3(s), and C6.11.3(t) of the CHBDC (2014), S6.1-14 (Commentary).

10.4.4 Pile Installation

All piles shall be installed in accordance with OPSS 903.

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

Pile driving must be controlled in accordance with Standard Provision SS103-11 (Hiley Formula) and an ultimate pile resistance must be specified by the designer. The Hiley formula does not need to be used until the pile tip is within 2 m of the design tip elevation. The appropriate pile driving note to be shown on the contract drawing is "Piles to be driven in accordance with Standard SS103-11 using an ultimate geotechnical resistance of R kN per pile" where "R" must have a minimum value of twice the factored design load at ULS. It is recommended that Pile Driving Analysis (PDA) testing be conducted on a minimum of 50 % of the piles per foundation element in conjunction with the Hiley tests at this site, to ensure the integrity of the pile and to verify pile ultimate geotechnical resistance.

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

Auger grinding was noted during drilling in the sand and silt till deposit. 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).

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 G. 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.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. The upper 3 m of the pile will lie within the stiff to very stiff approach embankment fill or the underlying hard till. Accordingly, to provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP as specified by the integral abutment design procedures.

After the pile is driven, the space between the pile and the CSP should be filled with sand.

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 sand and silt till. The recommended foundation for the pier is spread footing founded on very dense native silty sand.
- For non-integral abutments, it is recommended that the abutments and pier be supported on spread footings founded on native undisturbed compact to very dense silty sand and hard silty clay till.

11. RETAINING WALLS

The GA drawing dated July 2012 includes construction of one retaining wall, on the south side of the west abutment to retain the southwest approach embankment fill. The retaining wall will extend towards Wellington Street and will be approximately 22.5 m long and 3.1 m high. It is understood that the current design calls for an RSS wall.

RSS walls used on this project must be specified to be “High Performance” and “High Appearance”. Therefore, it is important that the RSS walls be founded on soil capable of supporting the imposed loading and limiting settlements under the RSS wall to acceptable magnitudes.

Provided the RSS design takes into account the subsurface conditions at this site and proper foundation preparation is carried out prior to construction of the walls, RSS systems are expected to meet the aesthetic and structural requirements.

To provide an acceptable foundation performance, the RSS must be founded on the hard silty clay till. The highest recommended base levels for the underside of the RSS system are as presented in Table 11.1.

Table 11.1 – Founding Strata and Elevations

Retaining Wall	Borehole	Highest Recommended Founding Elevation	Founding Stratum	Factored ULS (kN)	SLS (kN) (up to 25 mm settlement)
RW-08 South side of west abutment	NE16-14 RW08-01	320.0	Very stiff to hard silty clay till	350	250

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

If required, the RSS may be founded on engineered fill founded on the native, very stiff to hard silty clay till. Engineered fill placed under the RSS mass to achieve the design founding level

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must consist of OPSS Granular “A” compacted to 100% of its SPMDD at a moisture content within 2% of optimum. The engineered pad must extend at least 500 mm beyond the limits of the RSS mass and levelling strip.

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 (2014) Clauses 6.10.3 and 6.10.4.

As per MTO RSS Design Guidelines, the top of the levelling pad should be placed at least 0.5 m below finished grade (40% of frost depth in front of the wall).

A resistance factor of 0.6 m should be applied for cohesive soils as indicated in Table 6.2 of CHBDC (2014).

The entire block of reinforced earth must be designed against various modes of failure including sliding and overturning. Sliding resistance along the base of the wall or engineered granular fill in contact with the hard silty clay may be estimated using an ultimate friction coefficient of 0.4.

Topsoil, organics, fill, and any soft/wet material must be stripped from the footprint of the RSS. The subgrade under the RSS 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 RSS wall and placement and compaction of the granular fill must be carried out in the dry.

The proprietary RSS system must meet MTO’s specifications for performance and appearance. The RSS supplier/designer may specify more stringent criteria or other requirements related to the particular design. The internal stability of the RSS wall must be analyzed by the supplier/designer of the proprietary product selected for this site.

Lateral earth pressures acting on the walls should be computed as describe1d in Section 12. If the wall is retaining sloping backfill, appropriate earth pressure parameters for sloping backfill should be used.

Reference should be made to MTO RSS Design Guideline (2008) and, the TAC Design, Construction, Maintenance and Inspection Guide for MSE Walls (2017) for design and construction of retaining wall structures.

RSS walls must be constructed in accordance with MTO RSS SP 599S22 and SP 599S23.

11.1 Slope Stability of the Retained Soil System

A preliminary analysis of the global stability of the RSS wall was conducted to assess stability of a maximum 3.1 m high wall founded on the native very stiff to hard silty clay till, with a 2H:1V forward slope on the downslope side.

For the purpose of embankment stability analyses a commercially available slope stability program GEO-SLOPE was used. The Morgenstern-Price method was employed. The stability of the RSS wall was also checked under seismic loading assuming an acceleration of 0.097g. The computed factors of safety are as shown in Table 11.2. Slope stability computation outputs are included in Appendix F.

Table 11.2 Computed Factors of Safety

Condition	Factor of Safety	Figure (Appendix F)
RSS wall up to 3.1 m high at the west abutment		
Static Drained	1.5	1F
Static Undrained	2.2	2F
Seismic = 0.097g	1.8	3F

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. In the case of static loading, the factors of safety against global failure were 1.5 for drained conditions and 2.0 for undrained conditions. Under the estimated seismic loading, the minimum factor of safety calculated was 1.8. These factors of safety are considered to be acceptable for the proposed embankment bearing on the soils encountered at this site.

11.2 Settlement of the Retained soil system

The construction of a maximum 3.1 m high RRS wall on a 0.5 m thick pad of granular engineered fill will induce settlement in the underlying silty clay till.

The settlement was assessed using elastic methods. Based on these analyses, the settlement is estimated to be 25 mm to 30 mm.

This settlement will be immediate and essentially complete when construction of the RSS wall at the west abutment is completed.

Inspection of the RSS 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 2014 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 2014, a compaction surcharge should be added. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS.PROV 501.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 12.1.

Table 12.1 – Earth Pressure Coefficients

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

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used.

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

The factors in Table 12.1 are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to be used in the design can be estimated from Figure C6.16 in the Commentary to the CHBDC 2014.

It is recommended that perforated sub-drains and/or weep holes be installed, where applicable, to provide positive drainage of the granular backfill behind the abutment walls. Reference may be made to OPSD 3102.100 where appropriate.

13. APPROACH EMBANKMENTS

Based on the GA drawing dated July 2012, the proposed finished grade at the structure will be at about Elevations 331.5 to 330.3 at the west and east abutments, respectively. The existing ground surface at the site, varies from 322.3 to 324.6. As a result, placement of new fill of approximately 10.9 m will be required for the west approach of the proposed N-E ramp and approximately 6.3 m for the east approach.

All embankment fill must be constructed with adequate quality control in accordance with OPSS.PROV 206 and OPSS.PROV 501 requirements. OPSS>PROV>1010 Granular B Type I or SSM materials should be used.

It is also recommended that all permanent and temporary slope surfaces be vegetated and seeded in accordance with current MTO practice with reference to OPSS.PROV 804. It is important to note that slopes steeper than 2H:1V may be subject to surficial instability which may include sloughing and gullying. Surface runoff and precipitation must be prevented from flowing perpendicularly down any slope surface. Erosion protection measures will have to be taken as necessary to maintain slope stability.

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

13.1 Slope Stability of Side Slope

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 earth fill will have stable side slopes at inclinations of up to 2H:1V.

Where earth fill embankments are higher than 8 m, mid-height berms should be incorporated in each 8 m vertical interval. The berms should:

- extend for the length through which the embankment height exceeds 8 m
- be at least 2 m wide
- have 2% positive grade to shed run-off water

The analyses of global stability for the new forward slope configuration including the RSS wall, was presented in Section 11. In this section of the report typical sideslope configuration was analysed.

The Morgenstern-Price method was employed in conjunction with a commercially available slope stability program GEO-SLOPE to carry out the analyses. The computed factors of safety are as shown in Table 13.1. Graphical outputs of these analyses are included in Appendix F.

Table 13.1 Computed Factors of Safety

Condition	Factor of Safety	Figure (Appendix F)
Side Slope		
Static Drained	1.8	4F
Static Undrained	2.3	5F
Seismic = 0.097g	1.7	6F

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 effective stress (drained) conditions. In the case of static loading, the factors of safety against global failure were 1.8 for drained conditions, and 2.3 for undrained conditions. Under the estimated seismic loading, the minimum factor of safety calculated was 1.7. These range of factors of safety are considered to be acceptable for this site.

13.2 Settlement

It is estimated that at the approach embankments, settlements of 25 mm to 30 mm will occur in the foundation soils under the loading imposed by approximately 10.2 m of the new approach fill. This settlement will be immediate and essentially complete when construction of the fill is completed.

No long term settlement or global stability issues are anticipated for approach embankments built at this site.

14. 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 sand fill, native hard silty clay till, and into the native very dense silty sand.

For the purposes of the OHSA, the fill and native soils (silty sand) above the water table are classified as Type 3. Cohesionless soils below the water table are classified as Type 4.

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.

15. 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 Select Subgrade Material (SSM) 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.

16. GROUNDWATER AND SURFACE WATER CONTROL

The groundwater levels measured in the piezometers ranged from 3.8 m and 14.1 m below the ground surface (Elevations 319.4 to 309.1). Seasonal fluctuations of the groundwater level are to be expected. Excavation for footing or pile cap construction may extend below the groundwater level at some locations.

Temporary excavation for footing/pile cap construction will extend below the measured groundwater levels. Also, seepage perched water from the granular fill 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 base boiling and side wall sloughing of the soil below the water table making it difficult to maintain a dry, sound base on which to work.

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 fill	5.6×10^{-3} to 1.96×10^{-2}
Silty sand	2.3×10^{-6} to 4.9×10^{-5}
Silty clay/Silty clay till	1×10^{-8}
Sand and silt till	2.3×10^{-6}

Dewatering of all excavations should be carried out in accordance with OPSS. PROV 517, SP 517F01 Amendment to OPSS 517, November 216 (issued July 2017), and OPSS. PROV 902.

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

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

17. ROADWAY PROTECTION

If roadway protection is required during construction of the proposed ramp, an item titled "Protection System" as per OPSS 539 should be included in the contract documents. It is recommended that Performance Level 2 as per Clause 539.04.01.01 and the alignment of the shoring be specified on the contract drawings.

The design of roadway protection should be the responsibility of the Contractor. However, one option that is considered to be suitable for use as temporary shoring at this site is a soldier pile and lagging wall.

A temporary soldier pile and lagging wall may be designed using the parameters given below:

γ	=	20 kN/m^3
γ_w	=	10 kN/m^3
K_a	=	0.33 (approach fills)
	=	0.33 (silty clay/silty clay till)
	=	0.31 (silty sand)
K_p	=	3.0 (approach fills)
	=	3.0 (silty clay/silty clay till)
	=	3.2 (silty sand)

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. All shoring systems should be designed by a Professional Engineer experienced in such designs.

18. SEISMIC CONSIDERATIONS

In accordance with the CHBDC 2014, the selection of the seismic site classification is based on the averaged soil conditions encountered in the upper 30 m of the stratigraphy. The stratigraphy of the site consists of topsoil and fill overlaying a layer of silty clay till and, compact to very dense silty sand. Below the silty sand, a layer of very stiff to hard silty clay was encountered, which was underlain by very dense sand and silt till. This would correspond to a Seismic Site Class D in accordance with Table 4.1, Clause 4.4.3.2 of the CHBDC. The peak ground acceleration, PGA, for a 2% in 50-year probability of exceedance at this site is 0.075 g as per the National Building Code of Canada (NBCC). Since this site is classified as Class D, the factored PGA for a 2% in 50-year probability of exceedance at this site is 0.097 g.

In accordance with Clause 4.6.5 of the CHBDC 2014, 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 18.1 may be used:

Table 18.1 – 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

The site is underlain by typically compact to very dense silty sand overlying very stiff to hard silty clay and very dense silt and sand till, and liquefaction is not considered to be a concern at this site.

19. ADJACENT STRUCTURES 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 replacement structures and associated works. Protection and/or relocation of utilities may be required. Underground utilities should not be undermined or damaged during new foundation construction.

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 (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 operating structure during foundation construction to monitor if there is any movement or distress.
- The structural designers should assess the magnitude of settlement or horizontal displacement that would constitute a concern for the stability or serviceability of the existing operational structures. These limits should be incorporated into the monitoring program as review and alert levels.

20. CORROSION AND SULPHATE ATTACK POTENTIAL

The results of the corrosivity and sulphate analytical tests conducted on a sand fill sample 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 very mild.
- Appropriate protection measures commensurate with the above are recommended if metal structural elements are used. The effects of road de-icing salts should be also considered.

21. CONSTRUCTION CONCERNS

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

1. Pile Installation

Although there was little direct evidence of their presence during drilling, glacial till deposits inherently contain boulders. It is possible that a pile will achieve refusal at a higher elevation than anticipated due to encountering a boulder.

2. 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.

3. Groundwater Control

   Seepage and perched groundwater may be encountered within the cohesionless fill and native cohesionless soils. 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 excavation. All footings/pile caps must be constructed in the dry.

22. CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. R. Palomeque Reyna, P.Eng.

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

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Appendix A

Record of Borehole Sheets

Ministry of Transportation Ontario

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RECORD OF BOREHOLE No NE16-13

METRIC

GWP# 408-88-00 LOCATION N-E Ramp over Wellington Street, MTM NAD 83 Zone 10: N 4 814 511.2 E 226 270.2 ORIGINATED BY MB

DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MP

DATUM Geodetic DATE 2018.05.01 - 2018.05.01 LATITUDE 43.466395 LONGITUDE -80.470538 CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
320.6	GROUND SURFACE				
0.0	**TOPSOIL**, occasional rootlets				
320.3 / 0.3	Loose, Dark Brown, Moist	1	SS	10	
320.0 / 0.7	**SAND**, some silt, Loose, Brown, Moist (FILL)	2	SS	12	0 14 58 28
	Silty **CLAY**, some sand, Stiff to Hard, Brown, Moist (TILL)	3	SS	32	
318.4 / 2.2	Silty **SAND**, some clay, Dense to Very Dense, Brown, Moist	4	SS	45	
		5	SS	36	0 66 23 11
		6	SS	65	
		7	SS	67	
313.5 / 7.2	Silty **CLAY**, Hard, Brown, Moist				
312.7 / 7.9	Silty **SAND**, Dense, Brown, Moist	8	SS	44	
312.0 / 8.7		9	SS	51	

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (320, 319, 318, 317, 316, 315, 314, 313, 312, 311); 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

ONTMT4S2 MTO-11375.GPJ 2017TEMPLATE(MTO).GDT 1/24/19

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No NE16-13

METRIC

GWP# 408-88-00 | LOCATION N-E Ramp over Wellington Street, MTM NAD 83 Zone 10: N 4 814 511.2 E 226 270.2 | ORIGINATED BY MB

DIST | HWY 7 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY MP

DATUM Geodetic | DATE 2018.05.01 - 2018.05.01 | LATITUDE 43.466395 | LONGITUDE -80.470538 | CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
	Silty **CLAY** Hard Brown Moist						310				
			10	SS	42						
							309				
			11	SS	41		308				0 0 73 27
							307				
			12	SS	30						
306.3 14.3	END OF BOREHOLE AT 14.3m. BOREHOLE OPEN TO 14.3m UPON COMPLETION. MUD WAS ADDED DURING DRILLING; THEREFORE, IT WAS NOT POSSIBLE TO MEASURE THE WATER LEVEL UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.3m AND AUGER CUTTINGS TO SURFACE.										

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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No NE16-14

METRIC

GWP# 408-88-00 | LOCATION N-E Ramp over Wellington Street, MTM NAD 83 Zone 10: N 4 814 514.0 E 226 281.6 | ORIGINATED BY MB

DIST ____ HWY 7 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.24 - 2018.04.24 | LATITUDE 43.466421 | LONGITUDE -80.470398 | CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
321.5	GROUND SURFACE				
0.0	Clayey **SILT**, some sand to sandy, occasional cobbles, occasional rootlets, Firm, Brown, Moist (FILL)	1	SS	8	
320.8 / 0.7	**SAND**, some gravel, Compact, Brown, Wet (FILL)	2	SS	18	
320.0 / 1.4	Silty **CLAY**, some sand to sandy, trace gravel, Very Stiff to Hard, Wet (TILL)	3	SS	23	5 28 42 25
		4	SS	31	
		5	SS	30	
317.3 / 4.1	Silty **SAND**, some clay, trace gravel, Very Dense, Brown, Moist	6	SS	66	
315.2 / 6.2	Silty **CLAY**, Very Stiff to Hard, Brown, Moist	7	SS	40	
		8	SS	41	
		9	SS	85	0 0 72 28

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (321, 320, 319, 318, 317, 316, 315, 314, 313, 312); 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; STRAT PLOT.

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

ONTMT4S2 MTO-11375.GPJ 2017TEMPLATE(MTO).GDT 1/24/19

RECORD OF BOREHOLE No NE16-14

METRIC

GWP# 408-88-00 | LOCATION N-E Ramp over Wellington Street, MTM NAD 83 Zone 10: N 4 814 514.0 E 226 281.6 | ORIGINATED BY MB

DIST ___ HWY 7 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.24 - 2018.04.24 | LATITUDE 43.466421 | LONGITUDE -80.470398 | CHECKED BY RPR

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY** Very Stiff to Hard Brown Moist				311	
		10	SS	59	310	
		11	SS	23	309	
					308	
		12	SS	44	307	0 0 44 56
		13	SS	66	306	
305.2 16.3	**SAND** and **SILT**, some clay, trace gravel Very Dense Brown Wet (TILL) Auger grinding from 17.1 to 17.7m				305	
		14	SS	100/ 0.154	304	
		15	SS	100/ 0.100	303	
					302	

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

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

(%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No NE16-14

METRIC

GWP# 408-88-00 LOCATION N-E Ramp over Wellington Street, MTM NAD 83 Zone 10: N 4 814 514.0 E 226 281.6 ORIGINATED BY MB

DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MP

DATUM Geodetic DATE 2018.04.24 - 2018.04.24 LATITUDE 43.466421 LONGITUDE -80.470398 CHECKED BY RPR

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
301.3			16	SS	100/ 0.150						6 41 36 17
20.1	END OF BOREHOLE AT 20.1m. BOREHOLE OPEN TO 20.1m UPON COMPLETION. MUD WAS ADDED DURING DRILLING; THEREFORE, IT WAS NOT POSSIBLE TO MEASURE THE WATER LEVEL UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.3m AND AUGER CUTTINGS TO SURFACE.										

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No NE16-15

METRIC

GWP# 408-88-00 LOCATION N-E Ramp over Wellington Street, MTM NAD 83 Zone 10: N 4 814 524.2 E 226 315.5 ORIGINATED BY MB

DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MP

DATUM Geodetic DATE 2018.05.01 - 2018.05.02 LATITUDE 43.466517 LONGITUDE -80.469980 CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
323.2	GROUND SURFACE										
0.0	**TOPSOIL**, occasional rootlets Very Loose Dark Brown Moist		1	SS	2		323				
322.5 0.7	**SAND**, trace silt, trace clay Compact Brown Moist (FILL)		2	SS	14		322				
			3	SS	29		321				
			4	SS	21			0	92	5	3
319.9 3.3	Silty **SAND**, trace clay Compact to Very Dense Brown Moist		5	SS	21		320				
			6	SS	100/ 0.175		319 / 318				
			7	SS	100/ 0.225		317	0	70	24	6
			8	SS	87		316 / 315				
313.7 9.4	Silty **CLAY** Hard Brown		9	SS	44		314				

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

Continued Next Page

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

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

RECORD OF BOREHOLE No NE16-15

METRIC

GWP#	408-88-00	LOCATION	N-E Ramp over Wellington Street, MTM NAD 83 Zone 10: N 4 814 524.2 E 226 315.5	ORIGINATED BY	MB
DIST	HWY 7	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	MP
DATUM	Geodetic	DATE	2018.05.01 - 2018.05.02 LATITUDE 43.466517 LONGITUDE -80.469980	CHECKED BY	RPR

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	Silty **CLAY** Hard Brown Moist				313					
		10	SS	39	312					
		11	SS	33	311					
					310					
		12	SS	30	309					
		13	SS	36	308		0	0	37	63
					307					
		14	SS	31	306					
305.4 17.8	**SAND** and **SILT**, some gravel, some clay Very Dense Brown Moist (TILL)				305					
		15	SS	100/ 0.275	304					
	Auger grinding from 19.8m to 20.4m	16	SS	100/						

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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; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No NE16-15

METRIC

GWP# 408-88-00 LOCATION N-E Ramp over Wellington Street, MTM NAD 83 Zone 10: N 4 814 524.2 E 226 315.5 ORIGINATED BY MB

DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MP

DATUM Geodetic DATE 2018.05.01 - 2018.05.02 LATITUDE 43.466517 LONGITUDE -80.469980 CHECKED BY RPR

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "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/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
	SAND and **SILT**, some gravel, some clay Hard Brown Moist (TILL)				0.100		303				
							302				
301.5 21.7			17	SS	100/ 0.200				○		17 40 26 17

END OF BOREHOLE AT 21.7m.
WATER LEVEL AT 3.83m UPON COMPLETION.
Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.05.04	3.8	319.4
2018.05.16	13.6	309.6
2018.05.31	14.0	309.2
2018.06.25	14.1	309.1

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

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

RECORD OF BOREHOLE No NE16-16

METRIC

GWP# 408-88-00	LOCATION N-E Ramp over Wellington Street, MTM NAD 83 Zone 10: N 4 814 535.8 E 226 349.6		ORIGINATED BY GA/MB
DIST ___ HWY 7	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY MP
DATUM Geodetic	DATE 2018.04.13 - 2018.04.17	LATITUDE 43.466625 LONGITUDE -80.469561	CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
324.0	GROUND SURFACE						
0.0 / 0.1	**TOPSOIL:** (75mm)						
	SAND, trace gravel, trace silt, trace clay Compact Brown Moist (FILL)	1	SS	13		o	
		2	SS	19	323	o	
		3	SS	12	322	o	6 86 8 (SI+CL)
		4	SS	24		o	
321.0 / 3.0	Silty **SAND**, some gravel, trace clay Compact to Very Dense Brown Moist				321		
		5	SS	27	320	o	
		6	SS	76	319	o	
		7	SS	92	318	o	15 61 17 7
		8	SS	99	317 / 316	o	
		9	SS	100/ 0.125	315 / 314	o	

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; 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

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 NE16-16

METRIC

GWP# 408-88-00 LOCATION N-E Ramp over Wellington Street, MTM NAD 83 Zone 10: N 4 814 535.8 E 226 349.6 ORIGINATED BY GA/MB

DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MP

DATUM Geodetic DATE 2018.04.13 - 2018.04.17 LATITUDE 43.466625 LONGITUDE -80.469561 CHECKED BY RPR

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
	Silty **SAND** Very Dense Brown Wet	10	SS	100/ 0.225	313				
312.3 11.7	Silty **CLAY** Hard Brown Moist				312				
		11	SS	72	311				
		12	SS	45	310	0	0	41	59
					309				
		13	SS	48	308				
		14	SS	46	307				
					306				
		15	SS	53	305	0	0	37	63
304.6 19.4	**SAND** and **SILT**, some clay, trace gravel, occasional cobbles Very Dense (TILL)	16	SS	100/	304				

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DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

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

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

RECORD OF BOREHOLE No NE16-16

METRIC

GWP#	408-88-00	LOCATION	N-E Ramp over Wellington Street, MTM NAD 83 Zone 10: N 4 814 535.8 E 226 349.6	ORIGINATED BY	GA/MB
DIST	HWY 7	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	MP
DATUM	Geodetic	DATE	2018.04.13 - 2018.04.17 LATITUDE 43.466625 LONGITUDE -80.469561	CHECKED BY	RPR

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	Continued From Previous Page													
	SAND and **SILT**, some clay, trace gravel, occasional cobbles Very Dense Brown Wet (TILL) Auger grinding at 20.1m				0.137									
							303							
			17	SS	100/ 0.025				o		9	40	35	16
							302							
300.8			18	SS	100		301		o					
23.2	END OF BOREHOLE AT 23.2m. BOREHOLE OPEN TO 23.2m AND DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.3m AND AUGER CUTTINGS TO SURFACE.													

Legend: 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); REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No NE16-17

METRIC

GWP# 408-88-00 | LOCATION N-E Ramp over Wellington Street, MTM NAD 83 Zone 10: N 4 814 539.9 E 226 361.5 | ORIGINATED BY GA

DIST | HWY 7 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.12 - 2018.04.12 | LATITUDE 43.466663 | LONGITUDE -80.469415 | CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m3	GR	SA	SI	CL
324.1	GROUND SURFACE													
0.0	**TOPSOIL**: (50mm)						324							
0.0	**SAND**, some silt and clay, trace gravel Compact to Very Dense Brown Moist (FILL)		1	SS	12									
			2	SS	24		323				8	80	12 (SI+CL)	
			3	SS	65		322							
321.9 2.2	Silty **CLAY**, trace to some sand, trace gravel Very Stiff to Hard Brown to Grey Moist to Wet (TILL)		4	SS	24		321							
			5	SS	31		320							
319.5 4.6	Silty **SAND**, some gravel, trace clay Very Dense Brown Wet		6	SS	85		319				14	62	18	6
			7	SS	101		318 / 317							
	Brown to Grey		8	SS	89		316 / 315							
			9	SS	89									

Shear strength legend: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE. Water content legend: PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L. Scales: 20 40 60 80 100; water content 20 40 60.

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 NE16-17

METRIC

GWP# 408-88-00 LOCATION N-E Ramp over Wellington Street, MTM NAD 83 Zone 10: N 4 814 539.9 E 226 361.5 ORIGINATED BY GA

DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MP

DATUM Geodetic DATE 2018.04.12 - 2018.04.12 LATITUDE 43.466663 LONGITUDE -80.469415 CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	Continued From Previous Page													
	Silty **SAND** to **SAND** and **SILT**, some clay Very Dense Grey Wet						314							
			10	SS	68		313		o		0	47	43	10
312.8 11.3	END OF BOREHOLE AT 11.3m. BOREHOLE OPEN TO 11.3m AND WATER LEVEL AT 7.0m. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.													

SOIL PROFILE / SAMPLES / REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.04.27	7.2	316.9
2018.05.16	7.2	316.9
2018.05.31	7.0	317.1
2018.06.25	6.7	317.4

ONTMT4S2 MTO-11375.GPJ 2017TEMPLATE(MTO).GDT 1/24/19

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

15 20 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RW08-01

METRIC

GWP# 408-88-00	LOCATION Retaining Wall 8, MTM NAD 83 Zone 10: N 4 814 488.8 E 226 276.4		ORIGINATED BY MB
DIST HWY 7	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY MP
DATUM Geodetic	DATE 2018.05.01 - 2018.05.01	LATITUDE 43.466193 LONGITUDE -80.470459	CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
322.6	GROUND SURFACE				
0.0	**SAND**, some gravel, trace silt Loose to Compact Brown Moist (FILL)	1	SS	10	
		2	SS	18	
320.4 / 2.2		3	SS	22	
	Silty **CLAY**, some sand to sandy, trace gravel Hard Brown Moist (TILL)	4	SS	36	3 26 43 28
318.6 / 4.0		5	SS	34	
	Silty **SAND**, trace gravel, trace clay Very Dense Brown Moist	6	SS	48	
		7	SS	55	
		8	SS	98	0 60 31 9
314.4 / 8.2	Very Loose	9	SS	3	* Low SPT 'N' value due to soil ~~distubance during~~ drilling.

ELEVATION SCALE: 322, 321, 320, 319, 318, 317, 316, 315

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

END OF BOREHOLE AT 8.2m.
WATER LEVEL AT 6.06m UPON COMPLETION.
Well installation consists of 25mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.05.01	6.1	316.5
2018.05.16	6.1	316.5

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 RW08-01

METRIC

GWP# 408-88-00 LOCATION Retaining Wall 8, MTM NAD 83 Zone 10: N 4 814 488.8 E 226 276.4 ORIGINATED BY MB

DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MP

DATUM Geodetic DATE 2018.05.01 - 2018.05.01 LATITUDE 43.466193 LONGITUDE -80.470459 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
	2018.05.31 6.1 316.5										
	2018.06.25 5.8 316.8										

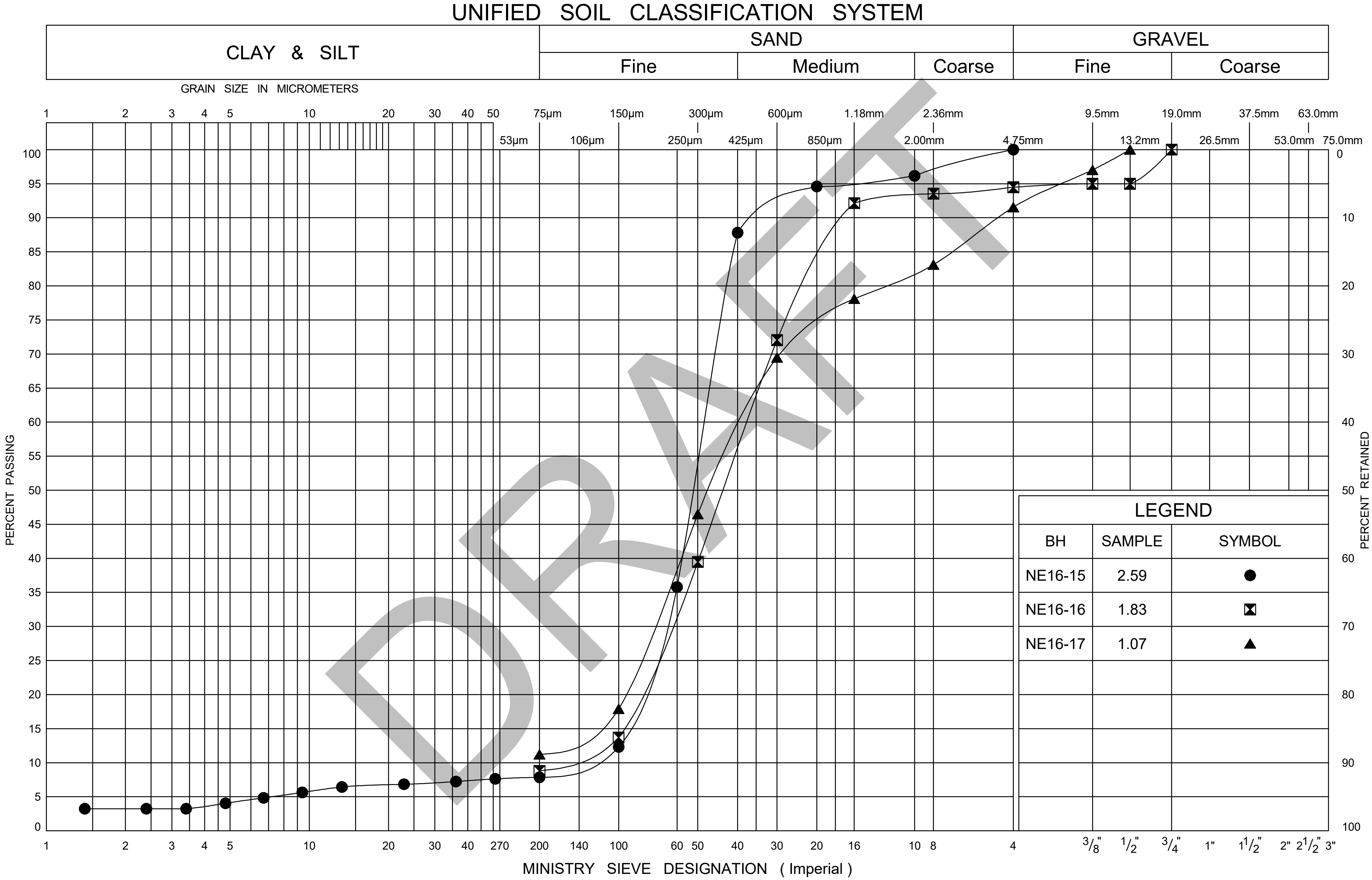
+3, ×3: Numbers refer to Sensitivity

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

ONTMT4S2 MTO-11375.GPJ 2017TEMPLATE(MTO).GDT 1/24/19

Appendix B

Laboratory Test Results and
Analytical Laboratory Test Results



LEGEND

BH	SAMPLE	SYMBOL
NE16-15	2.59	●
NE16-16	1.83	⊠
NE16-17	1.07	▲

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

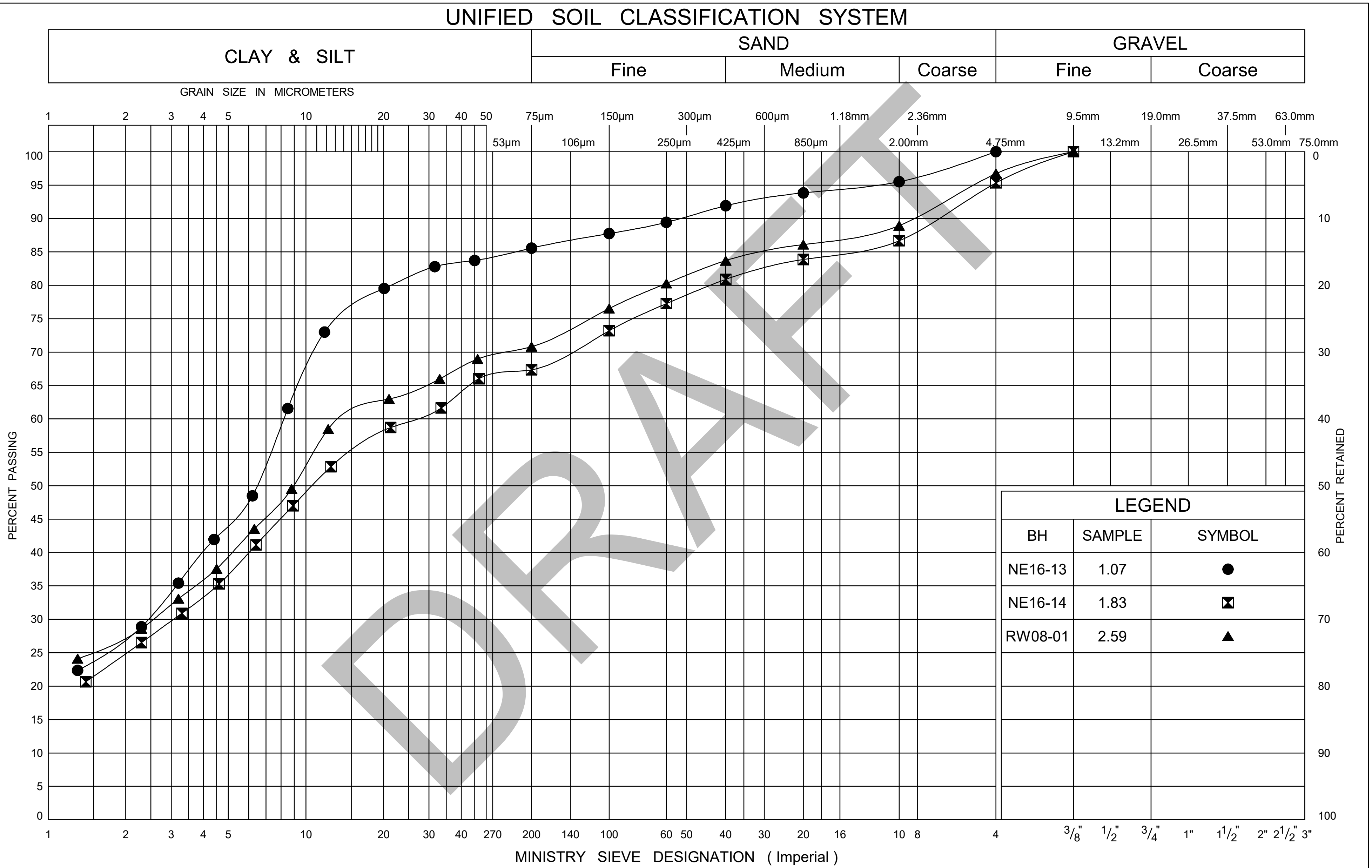
Sand Fill

FIG No B1

W P 408-88-00

N-E Ramp over Wellington Street

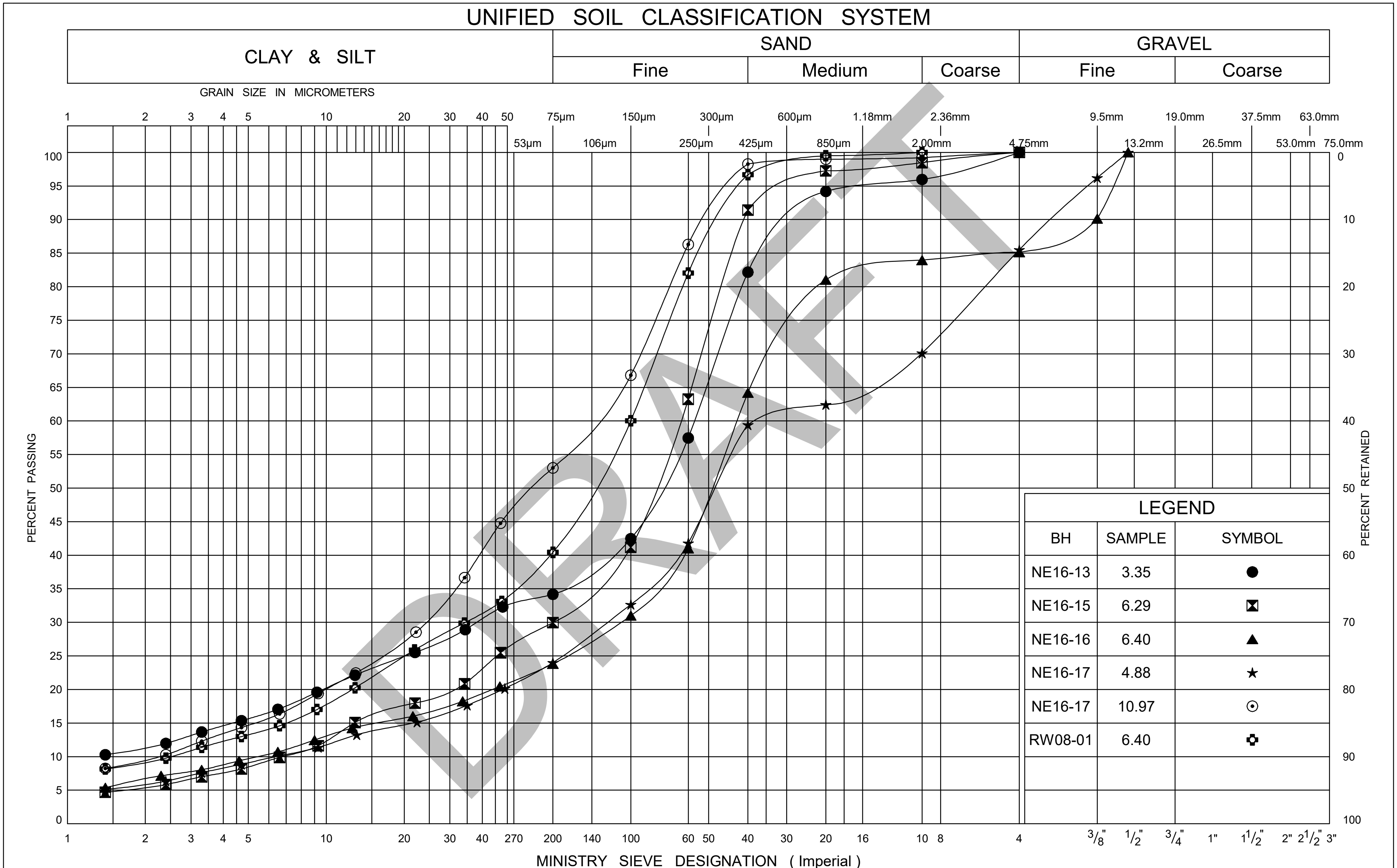
78 12 M



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION Silty Clay Till	FIG No B2 W P 408-88-00 N-E Ramp over Wellington Street

ONTARIO MOT GRAIN SIZE MTO-11375.GPJ ONTARIO MOT.GDT 10/15/18

78 12 M



Ministry of Transportation

Ontario

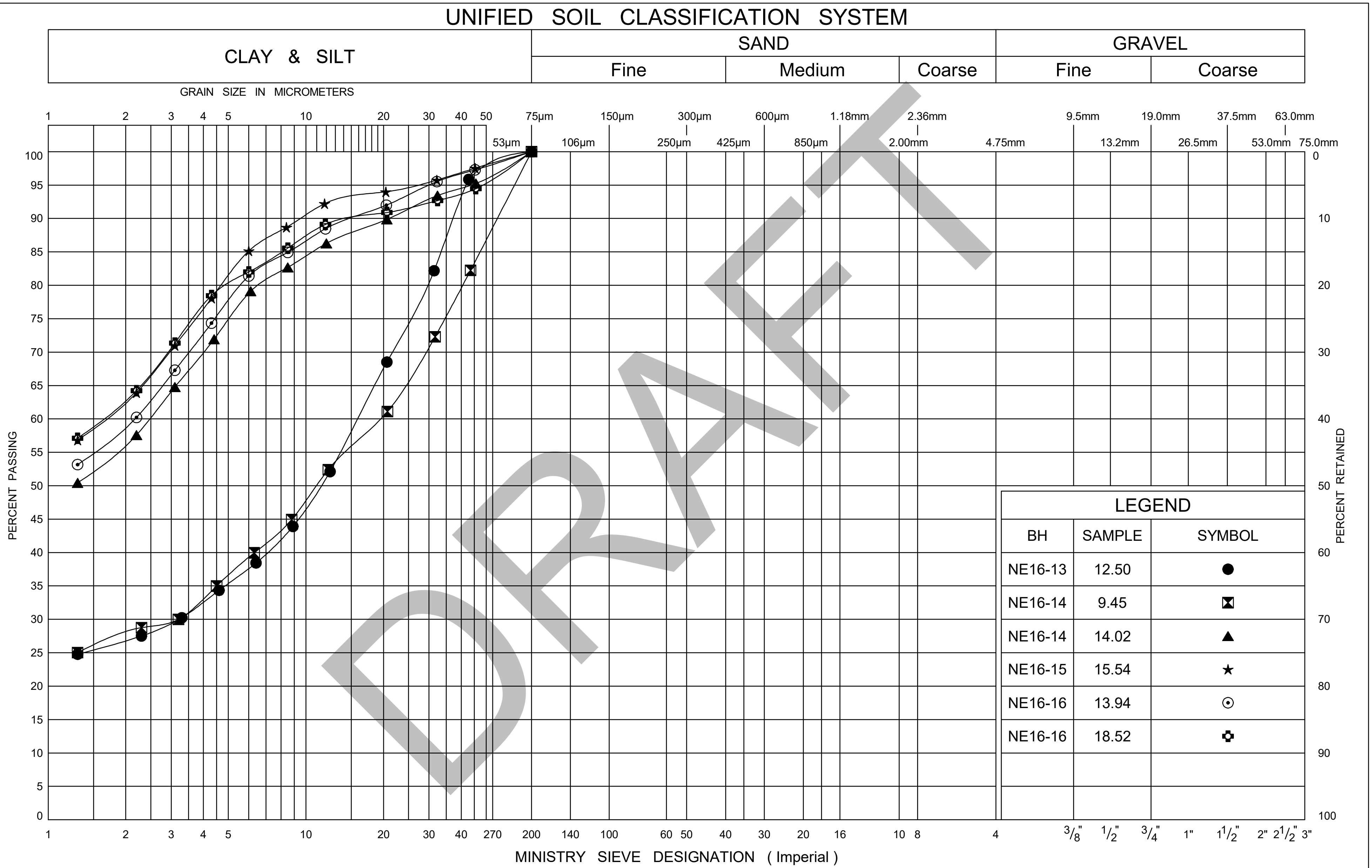
GRAIN SIZE DISTRIBUTION

Silty Sand, Sand

FIG No B3	
W P 408-88-00	
N-E Ramp over Wellington Street	

ONTARIO MOT GRAIN SIZE MTO-11375.GPJ ONTARIO MOT.GDT 8/28/18

78 12 M



Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION

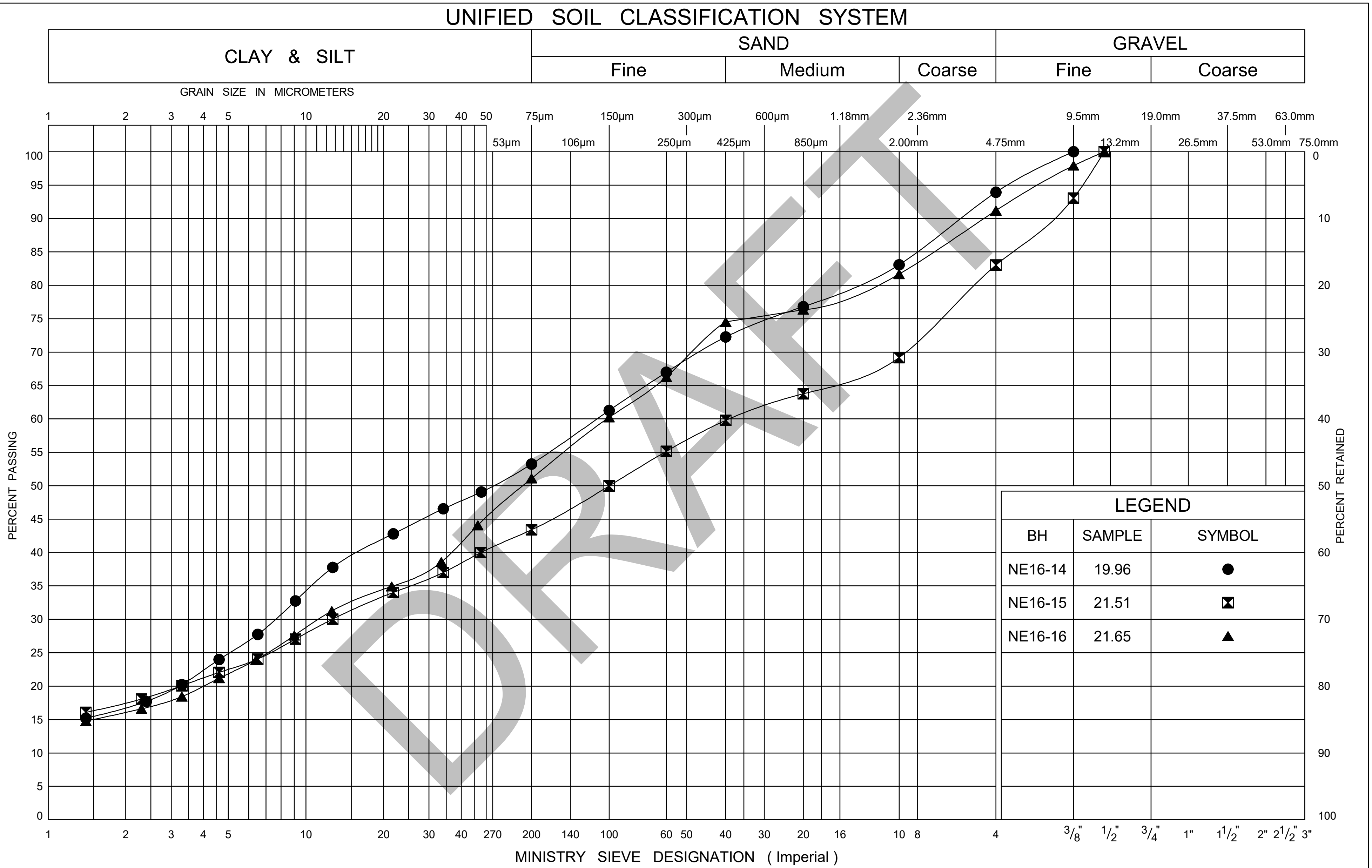
Silty Clay

FIG No B4

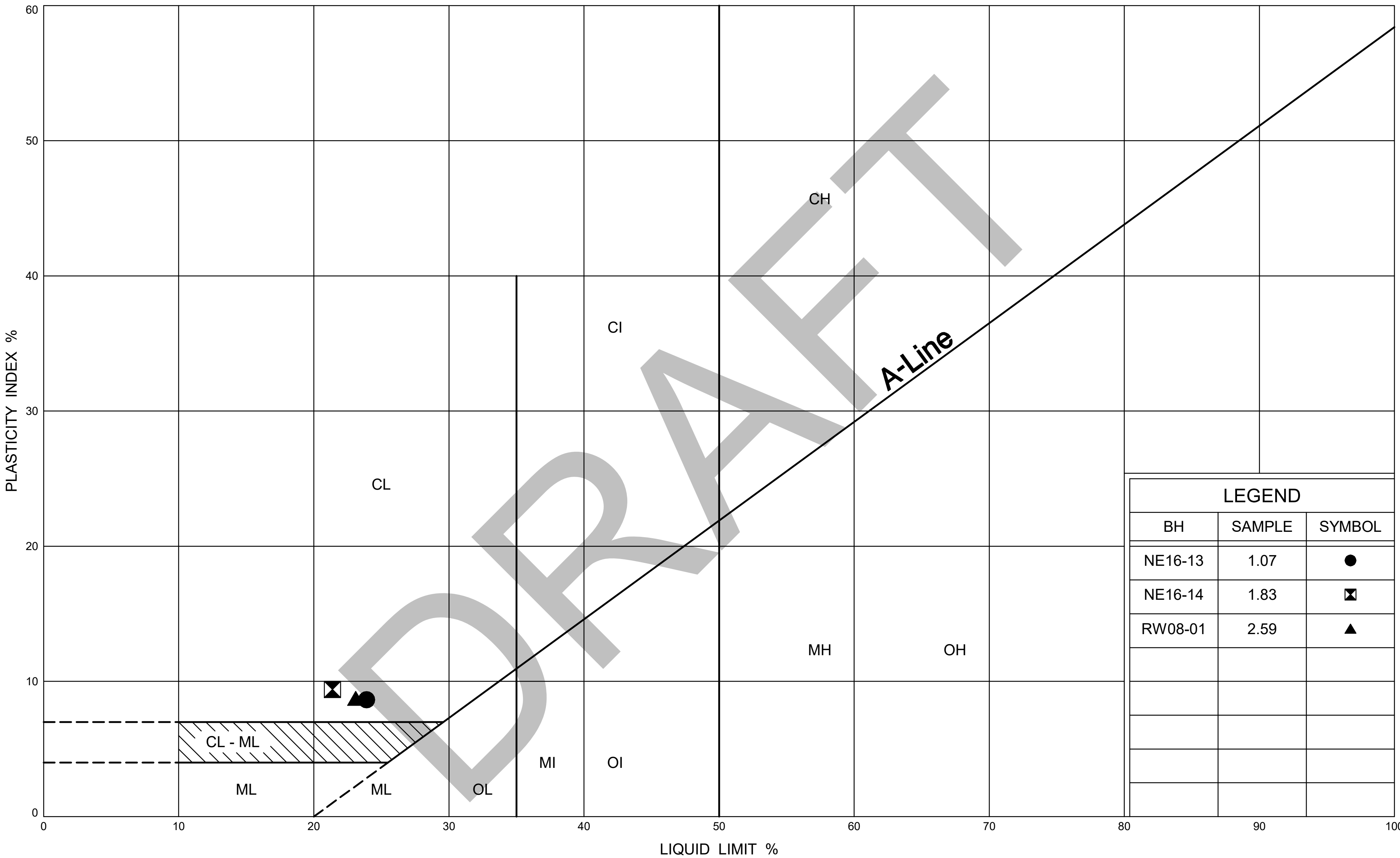
W P 408-88-00

N-E Ramp over Wellington Street

ONTARIO MOT GRAIN SIZE MTO-11375.GPJ ONTARIO MOT.GDT 10/15/18



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION Sand and Silt Till	FIG No B5
		W P 408-88-00
		N-E Ramp over Wellington Street



Ministry of Transportation
Ontario

PLASTICITY CHART

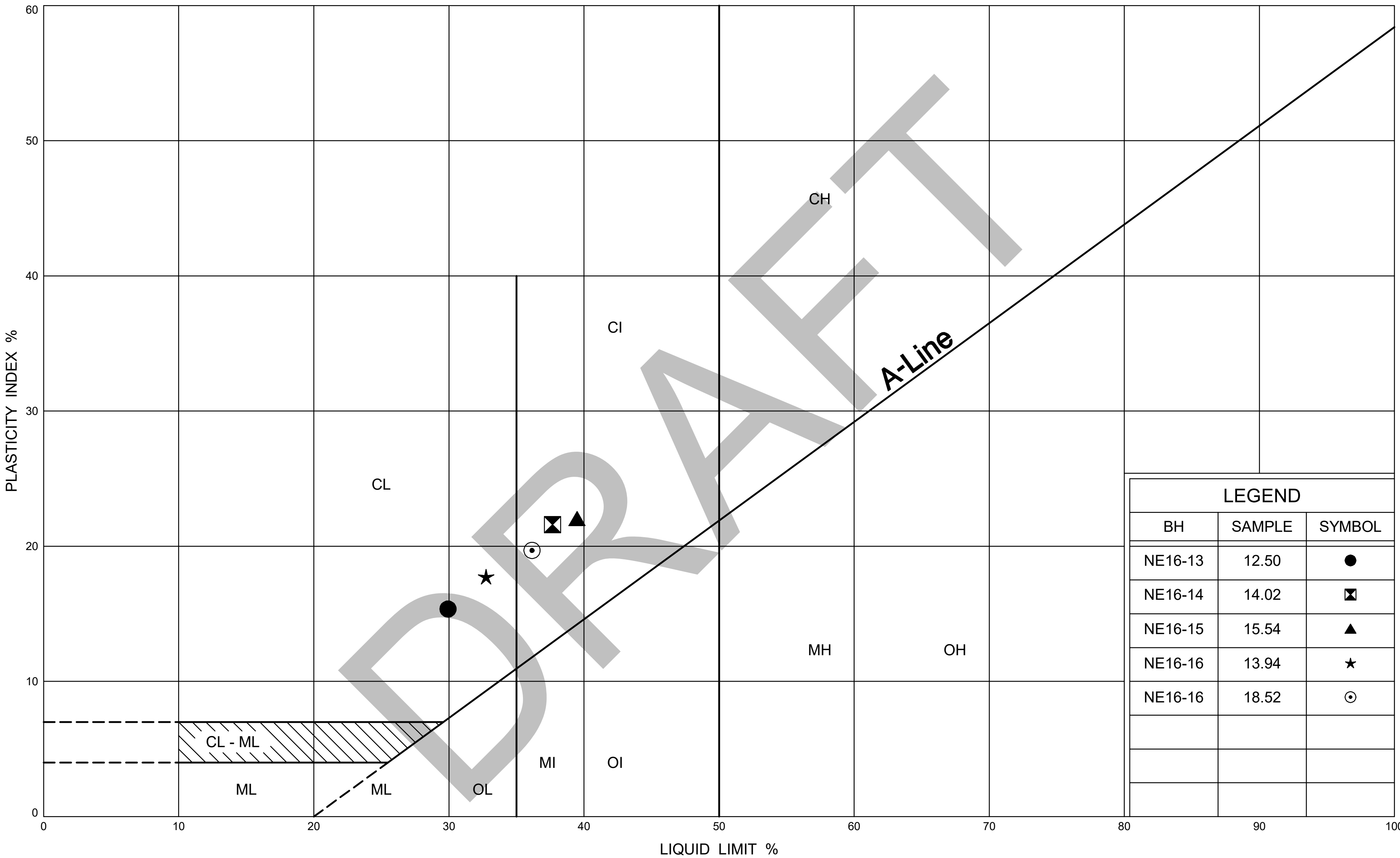
Silty Clay Till

FIG No B6

W P 408-88-00

N-E Ramp over Wellington Street

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
NE16-13	12.50	●
NE16-14	14.02	⧗
NE16-15	15.54	▲
NE16-16	13.94	★
NE16-16	18.52	⊙

ONTARIO MOT PLASTICITY CHART MTO-11375.GPJ ONTARIO MOT.GDT 10/15/18

Ministry of Transportation
Ontario

PLASTICITY CHART

Silty Clay

FIG No B7	
W P 408-88-00	
N-E Ramp over Wellington Street	

FINAL REPORT

CA14058-MAY18 R1

11375

Prepared for

Thurber Engineering Ltd.

FINAL REPORT

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	Thurber Engineering Ltd.	Project Specialist	Deanna Edwards, B.Sc, C.Chem
		Laboratory	SGS Canada Inc.
Address	103, 2010 Winston Park Drive Oakville, ON L6H 5R7.	Address	185 Concession St., Lakefield ON, K0L 2H0
Contact	Rocío Palomeque	Telephone	705-652-2000
Telephone	905-829-8666 x 263	Facsimile	705-652-6365
Facsimile		Email	deanna.edwards@sgs.com
Email	rreyna@thurber.ca	SGS Reference	CA14058-MAY18
Project	11375	Received	05/02/2018
Order Number		Approved	05/09/2018
Samples	Soil (7)	Report Number	CA14058-MAY18 R1
		Date Reported	05/09/2018

COMMENTS

Temperature of Sample upon Receipt: 8 degrees C

Cooling Agent Present: No

Custody Seal Present: No

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

Deanna Edwards, B.Sc, C.Chem

Deanna Edwards

TABLE OF CONTENTS

FINAL REPORT

CA14058-MAY18 R1

Client: Thurber Engineering Ltd.

Project: 11375

Project Manager: Rocío Palomeque

Samplers: N/A

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

			Sample Number	5	6	7	8	9	10	11
			Sample Name	RW12-05	RW10-04 SS4	RW 09-02 SS3	NE 16-16 SS4	RW13-01 SS4	SE16-05 SS3	SE16-06 SS5
			Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil
			Sample Date	20/04/2018	18/04/2018	11/04/2018	11/04/2018	11/04/2018	12/04/2018	23/04/2018
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**	**Result**	**Result**	**Result**	**Result**
Corrosivity Index										
Corrosivity Index	none	1		4	3	4	4	4	3	4
Soil Redox Potential	mV	-		230	182	274	164	133	232	215
Sulphide	%	0.02		< 0.02	< 0.02	< 0.02	< 0.02	< 0.02	< 0.02	< 0.02
pH	no unit	0.05		8.67	9.11	9.04	9.19	8.50	9.11	9.25
Resistivity (calculated)	ohms.cm	-9999		4610	17100	6670	13200	5250	13400	10100

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

			Sample Number	5	6	7	8	9	10	11
			Sample Name	RW12-05	RW10-04 SS4	RW 09-02 SS3	NE 16-16 SS4	RW13-01 SS4	SE16-05 SS3	SE16-06 SS5
			Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil
			Sample Date	20/04/2018	18/04/2018	11/04/2018	11/04/2018	11/04/2018	12/04/2018	23/04/2018
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**	**Result**	**Result**	**Result**	**Result**
General Chemistry										
Conductivity	uS/cm	2		217	59	150	76	190	75	99

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

			Sample Number	5	6	7	8	9	10	11
			Sample Name	RW12-05	RW10-04 SS4	RW 09-02 SS3	NE 16-16 SS4	RW13-01 SS4	SE16-05 SS3	SE16-06 SS5
			Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil
			Sample Date	20/04/2018	18/04/2018	11/04/2018	11/04/2018	11/04/2018	12/04/2018	23/04/2018
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**	**Result**	**Result**	**Result**	**Result**
Metals and Inorganics										
Moisture Content	%	0.1		9.3	4.4	11.3	8.3	13.4	4.1	8.8
Sulphate	µg/g	0.4		15	1.1	13	5.5	11	4.0	8.7

FINAL REPORT

CA14058-MAY18 R1

Client: Thurber Engineering Ltd.

Project: 11375

Project Manager: Rocío Palomeque

Samplers: N/A

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

			Sample Number	5	6	7	8	9	10	11
			Sample Name	RW12-05	RW10-04 SS4	RW 09-02 SS3	NE 16-16 SS4	RW13-01 SS4	SE16-05 SS3	SE16-06 SS5
			Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil
			Sample Date	20/04/2018	18/04/2018	11/04/2018	11/04/2018	11/04/2018	12/04/2018	23/04/2018
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**	**Result**	**Result**	**Result**	**Result**
Other (ORP)										
Chloride	μg/g	0.4		70	3.2	53	12	46	19	30

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	DIO0131-MAY18	µg/g	0.4	<0.4	6	20	95	80	120	106	75	125
Sulphate	DIO0131-MAY18	µg/g	0.4	<0.4	42	20	98	80	120	98	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	ECS0004-MAY18	%	0.02	<0.02	8	20	99	80	120			

pH

Method: SM 4500 | 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 (%) Low	Recovery Limits (%) High	Spike Recovery (%)	Recovery Limits (%) Low	Recovery Limits (%) High
pH	EWL0048-MAY18	no unit	0.05	NA	1		100			NA		

QC SUMMARY

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

SGS Environmental Services

Request for Laboratory Services and CHAIN OF CUSTODY

- Lakefield: 185 Concession St., Lakefield, ON K0L 2H0 Phone: 705-652-2000 Toll Free: 877-747-7658 Fax: 705-652-6365
- London: 657 Consortium Court, London, ON, N6E 2S8 Phone: 519-672-4500 Toll Free: 877-848-8060 Fax: 519-672-0361 Web: www.ca.sgs.com

No:

Page 1 of 1

Laboratory Information Section - Lab use only

Received By: Ismail
Received Date (mm/dd/yyyy): 05 02 18 (mm\dd\yy)
Received Time: 11:00 Am
Received By (signature):
Custody Seal Present: ☐ NO
Custody Seal Intact: ☐ NO
Cooling Agent Present: ☐ N
Temperature Upon Receipt (°C) 12, 11.1, 10.
LAB LIMS #: CA14058-May 18
8x3

REPORT INFORMATION

Company: Thurber Eng.
Contact: Rocio Palomeque Reyna
Address: 103-2010 Winston Park Dr
Oakville, ON L6H 5R7
Phone:
Fax:
Email: rreyna@thurber.ca

INVOICE INFORMATION

☒ (same as Report Information)
Company:
Contact:
Address:
Phone:
Email:

PROJECT INFORMATION

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

TURNAROUND TIME (TAT) REQUIRED

☐ Regular TAT (5-7days)

TAT's are quoted in business days (exclude statutory holidays & weekends). Samples received after 3pm or on weekends : TAT begins the next business day

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

PLEASE CONFIRM RUSH FEASIBILITY WITH SGS REPRESENTATIVE PRIOR TO SUBMISSION

Specify Due Date:
Rush Confirmation ID:

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

REGULATIONS

Regulation 153 (2011):
☐ Table 1 ☐ Table 2 ☐ Table 3 ☐ Table ____
☐ Res/Park ☐ Ind/Com ☐ Agri/Other
Soil Texture: ☐ Coarse ☐ Medium ☐ Fine

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

Sewer By-Law:
☐ Sanitary ☐ Storm
Municipality:

RECORD OF SITE CONDITION (RSC) ☐ YES ☐ NO

ANALYSIS REQUESTED

	SAMPLE IDENTIFICATION	DATE SAMPLED	TIME SAMPLED	# OF BOTTLES	MATRIX	PHC F1-F4 BTEX	O.Reg153 Metals (ICP & hydride metals)	☐Hg ☐B-HWS ☐Cr(VI)	O.Reg 153 VOCs	Corrosivity								COMMENTS: Field Filtered (F) Preserved (P)
1	RW12-05	April 20/18		1	Soil					✓								
2	RW10-04 SS4	April 18/18		1	"					✓								
3	RW09-02 SS3	April 11/18		1	"					✓								
4	NE16-16 SS4	April 13/18		1	"					✓								
5	RW13-01 SS4	April 9/13		1	"					✓								
6	~~[illegible]~~ SE16-05 SS3	April 12/18		1	"					✓								
7	ES16-06 - SS5	April 23		1	"													
8																		
9																		
10																		

Observations/Comments/Special Instructions

Sampled By (NAME):	Signature:	Date: __/__/__ (mm/dd/yy)	Pink Copy - Client
Relinquished by (NAME):	Signature:	Date: 01/05/2018 (mm/dd/yy)	Yellow & White Copy - SGS

Revision #: 1.0
Date of Issue: 01 June, 2014

SGS

SAMPLE INTEGRITY REPORT

Project Number: 11375

ONTARIO REGULATION 153/04

SGS Sample ID CA14058-May18

Date / Time Sampled Apr 11,12,18,19,20,23

Client Sample ID See CofC

ALL

Sample Submission General Sample Integrity Violations

Temperature >10 C upon receipt if not sampled same day	☐
No evidence of cooling trend initiated if sampled same day	☐
Chain of Custody not submitted	☐
Chain of Custody incomplete	☐
Chain of Custody not signed / dated	☐
Chain of Custody not a current version	☐
Bottles / Samples listed on CoC but not received	☐
Bottles / Samples received but not listed on the CoC	☐
Sample container received empty	☐

Sample Specific Sample Integrity Violations

Sample received past hold time	☐	☐	☐	☐	☐	☐	☐
Incorrect preservation (including no preservation where required)	☐	☐	☐	☐	☐	☐	☐
Headspace present in VOC vial (aqueous)	☐	☐	☐	☐	☐	☐	☐
Sample(s) received frozen	☐	☐	☐	☐	☐	☐	☐
Bottle(s) broken or damaged in transport	☐	☐	☐	☐	☐	☐	☐
Discrepancy between sample label and chain of custody	☐	☐	☐	☐	☐	☐	☐
Analysis requirements absent / unclear	☐	☐	☐	☐	☐	☐	☐
Missing or incorrect sample label(s)	☐	☐	☐	☐	☐	☐	☐
Inappropriate sample container used	☐	☐	☐	☐	☐	☐	☐
Insufficient number of bottles received	☐	☐	☐	☐	☐	☐	☐
Limited sample volume	☐	☐	☐	☐	☐	☐	☐
Insufficient sample volume	☐	☐	☐	☐	☐	☐	☐
Sample contains multiple phases	☐	☐	☐	☐	☐	☐	☐

Sediment Log

Groundwater samples contain visible sediment / particulate	☐	☐	☐	☐	☐	☐	☐
Groundwater contains greater than 1cm of sediment / particulate matter in bottle	☐	☐	☐	☐	☐	☐	☐

Additional Comments/Remarks:

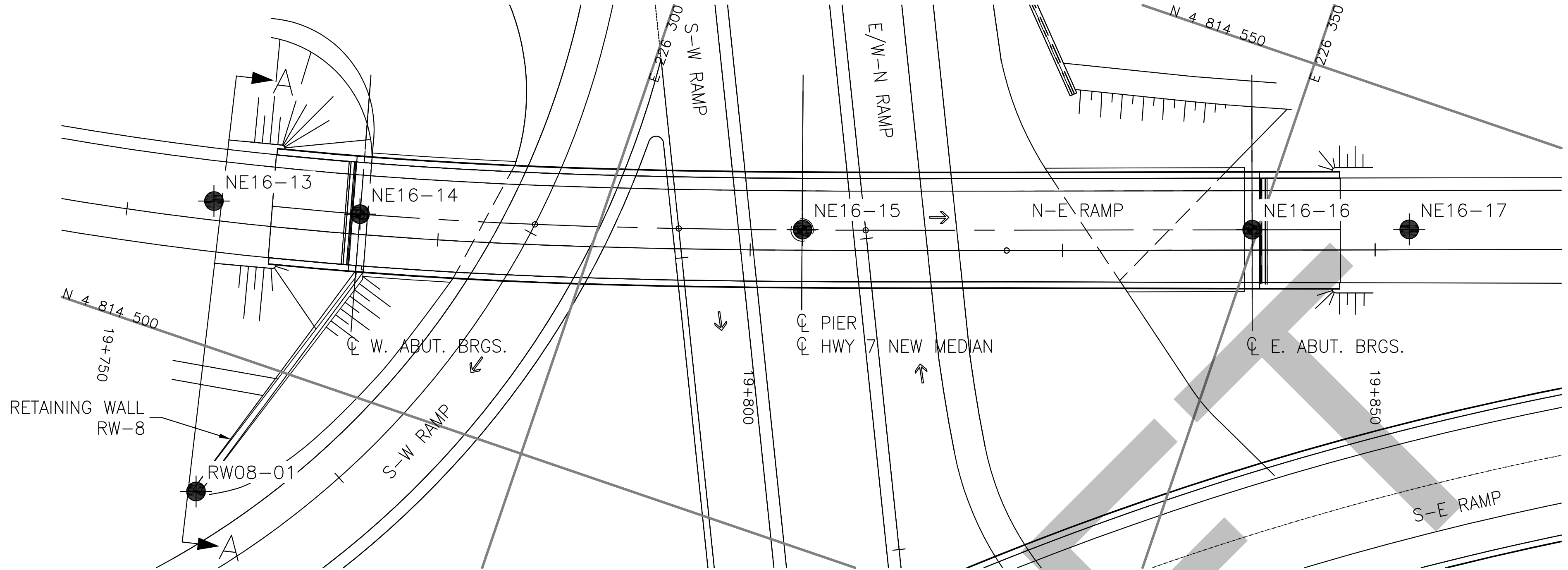
No issues upon receipt ☑

Initials: BM

THURBER

Appendix C

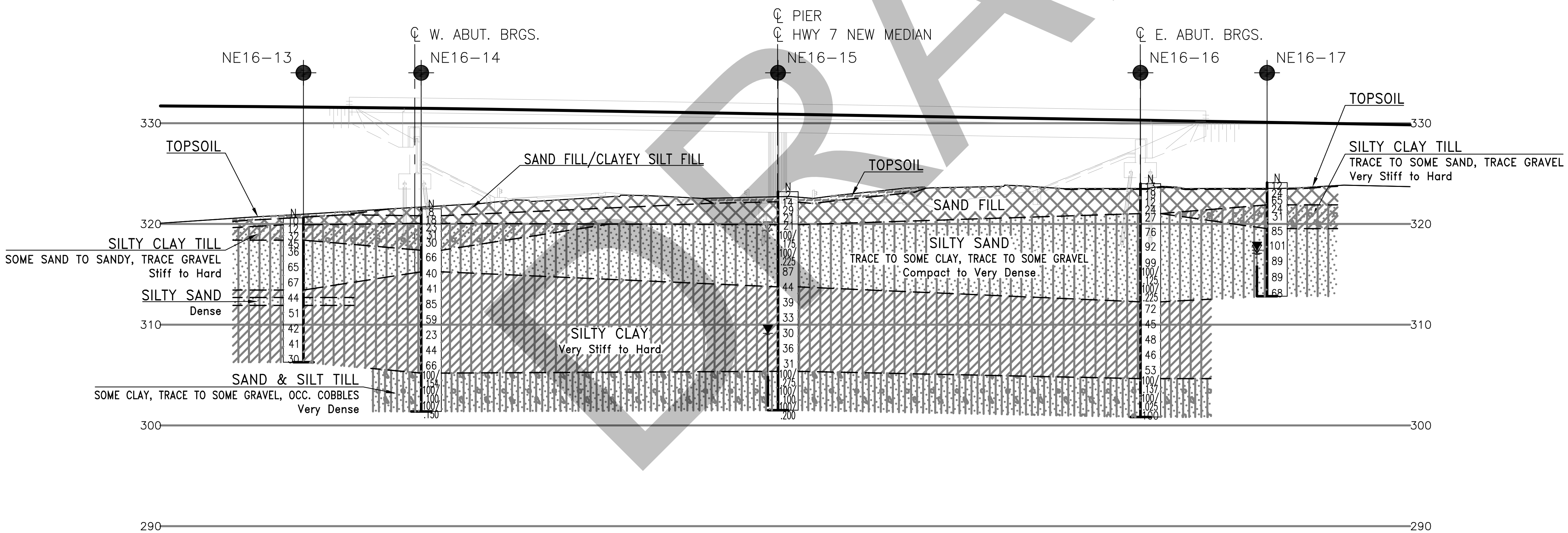
Borehole Locations and Soil Strata Drawing



PLAN

SCALE 1:500

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



PROFILE ALONG N–E RAMP OVER WELLINGTON STREET

SCALE 1:500

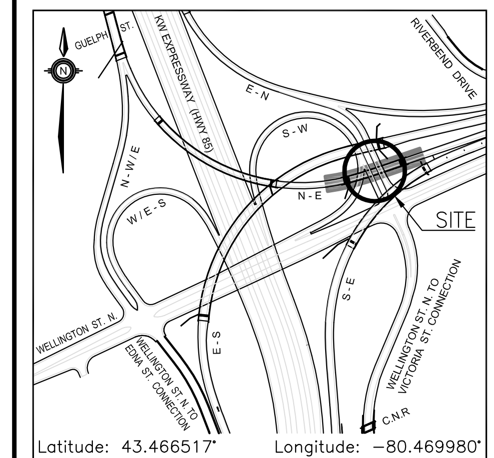
CONT No
GWP No 408–88–00

HIGHWAY 7
N–E RAMP OVER WELLINGTON ST
PROPOSED BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

wsp

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

	Borehole (Current Investigation)
	Borehole (2008 Investigation)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
NE16–13	320.6	4 814 511.2	226 270.2
NE16–14	321.5	4 814 514.0	226 281.6
NE16–15	323.2	4 814 524.2	226 315.5
NE16–16	324.0	4 814 535.8	226 349.6
NE16–17	324.1	4 814 539.9	226 361.5
RW08–01	322.6	4 814 488.8	226 276.4

-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.

REVISIONS | DATE | BY | DESCRIPTION

DESIGN RPR | CHK PKC | CODE | LOAD | DATE JAN 2019
DRAWN MFA | CHK RPR | SITE | STRUCT | DWG 1

FILENAME: H:\Drafting\10000\11375\ED–11375–BHPP–NEW.dwg
PLOTDATE: 1/24/2019 2:27 PM

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

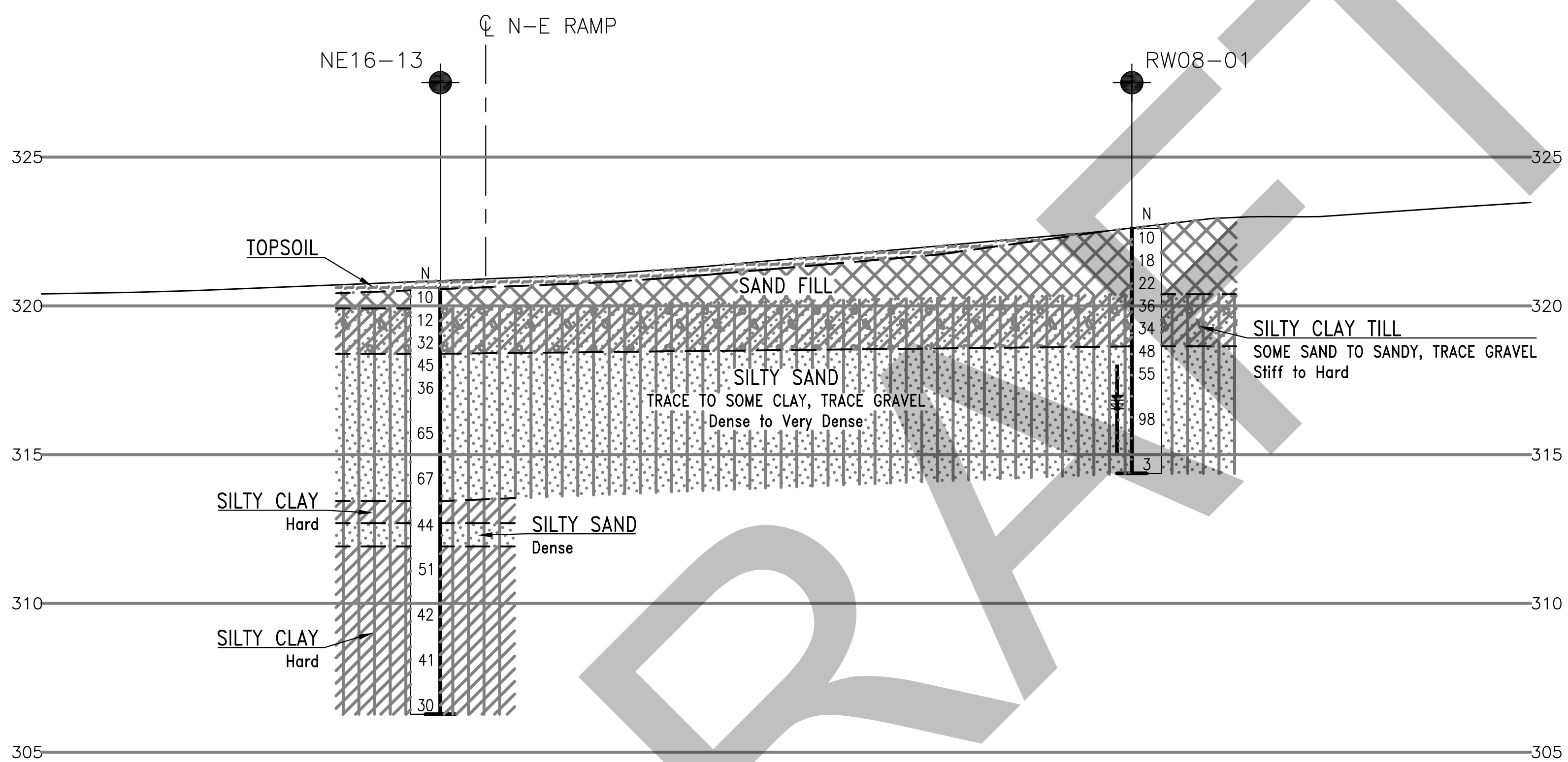
CONT No
GWP No 408–88–00

HIGHWAY 7
N–E RAMP OVER WELLINGTON ST
PROPOSED BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

wsp

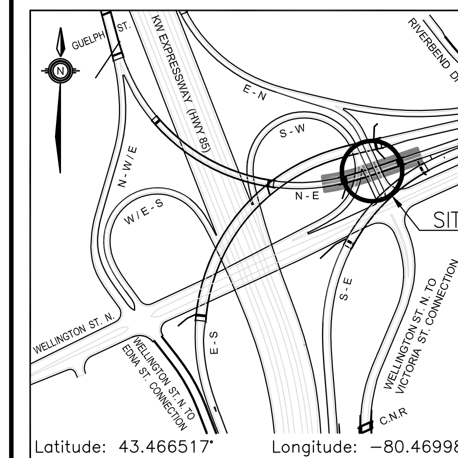
THURBER ENGINEERING LTD.



SECTION A–A

5 0 5 10m

SCALE 1:250



KEYPLAN

L E G E N D

Symbol	Description
	Borehole (Current Investigation)
	Borehole (2008 Investigation)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
NE16–13	320.6	4 814 511.2	226 270.2
NE16–14	321.5	4 814 514.0	226 281.6
NE16–15	323.2	4 814 524.2	226 315.5
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3) Coordinate system is MTM NAD 83 Zone 10.

GEOCRES No.

REVISIONS	DATE	BY	DESCRIPTION

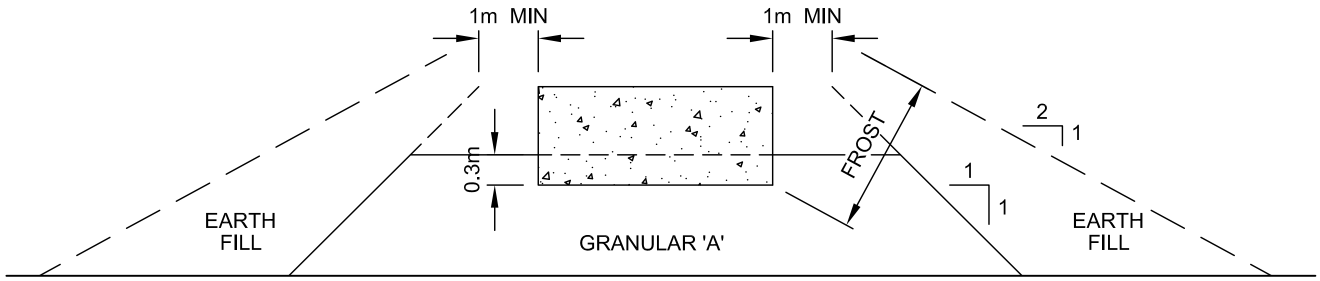
DESIGN RPR	CHK PKC	CODE	LOAD	DATE JAN 2019
DRAWN MFA	CHK RPR	SITE	STRUCT	DWG 2

FILENAME: H:\Drafting\10000\11375\TED-11375-BHPP-NEW.dwg
PLOTDATE: 1/24/2019 2:27 PM

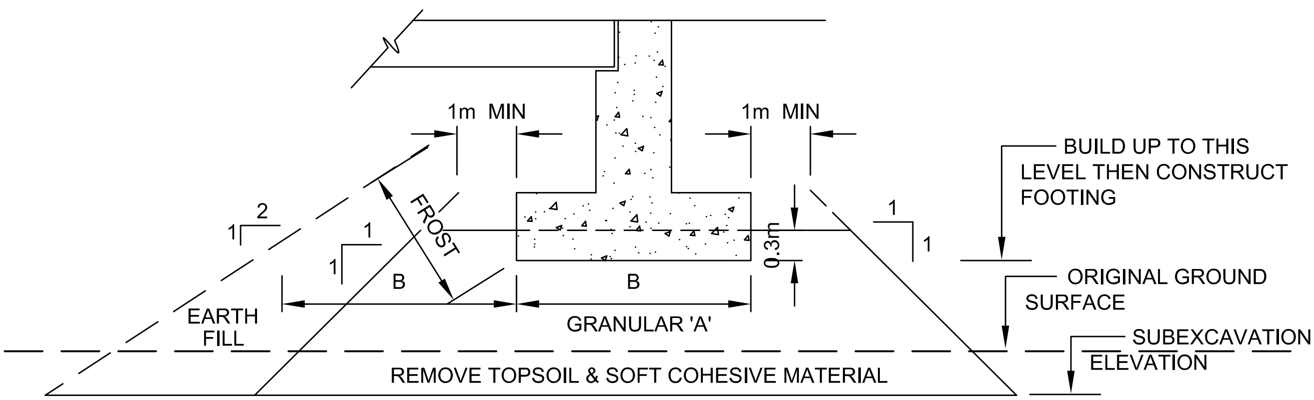
THURBER

Appendix D

Figure For
Engineered Fill Pad



CROSS-SECTION



LONGITUDINAL SECTION

NOTES:

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

ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE

THURBER ENGINEERING LTD.

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

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Appendix E

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. **FEASIBLE**	***Advantages:*** i. Generally less costly construction than deep foundation elements. ii. Better geotechnical resistance than spread footings on native soils. ***Disadvantages:*** i. Excavation (up to 3.0 m deep) 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:*** ii. High geotechnical resistance may be developed by driving the piles into very dense till iii. Comparatively short abutment stem possible iv. Permits integral abutment design. v. Readily installed. ***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. ***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. **NOT RECOMMENDED**
Pier	***Advantages:*** i. Generally less costly construction than deep foundation elements. ii. High geotechnical resistances available on the very dense native soils. ***Disadvantages:*** i. Dewatering will be required, depending on depth of excavation. **RECOMMENDED**	***Advantages:*** i. Generally less costly construction than deep foundation elements. ii. Better geotechnical resistance than spread footings on native soils. ***Disadvantages:*** i. 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. ***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. **FEASIBLE**	***Advantages:*** i. Construction of caissons could continue in freezing weather. ii. High geotechnical resistance available for units founded on very dense till. ***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. **NOT RECOMMENDED**

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Appendix F

Slope Stability Output

Project Number: 11375
Highway 7 - New
N-E Ramp over
S-W ramp, S-W to Wellington St.
ramp and E-N ramp
Retaining wall
Height: 3.1 m approx
Drained Analysis

Name: New embankment Fill Unit Weight: 19 kN/m³ Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1
Name: Stiff to hard silty clay fill Unit Weight: 19 kN/m³ Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1
Name: Very dense silty sand Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 ° Phi-B: 0 ° Piezometric Line: 1
Name: Very stiff to hard silty clay Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 31 ° Phi-B: 0 ° Piezometric Line: 1
Name: RSS Wall Unit Weight: 22 kN/m³ Cohesion: 200 kPa Phi: 45 ° Phi-B: 0 ° Piezometric Line: 1
Name: Granular pad Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 36 ° Phi-B: 0 ° Piezometric Line: 1

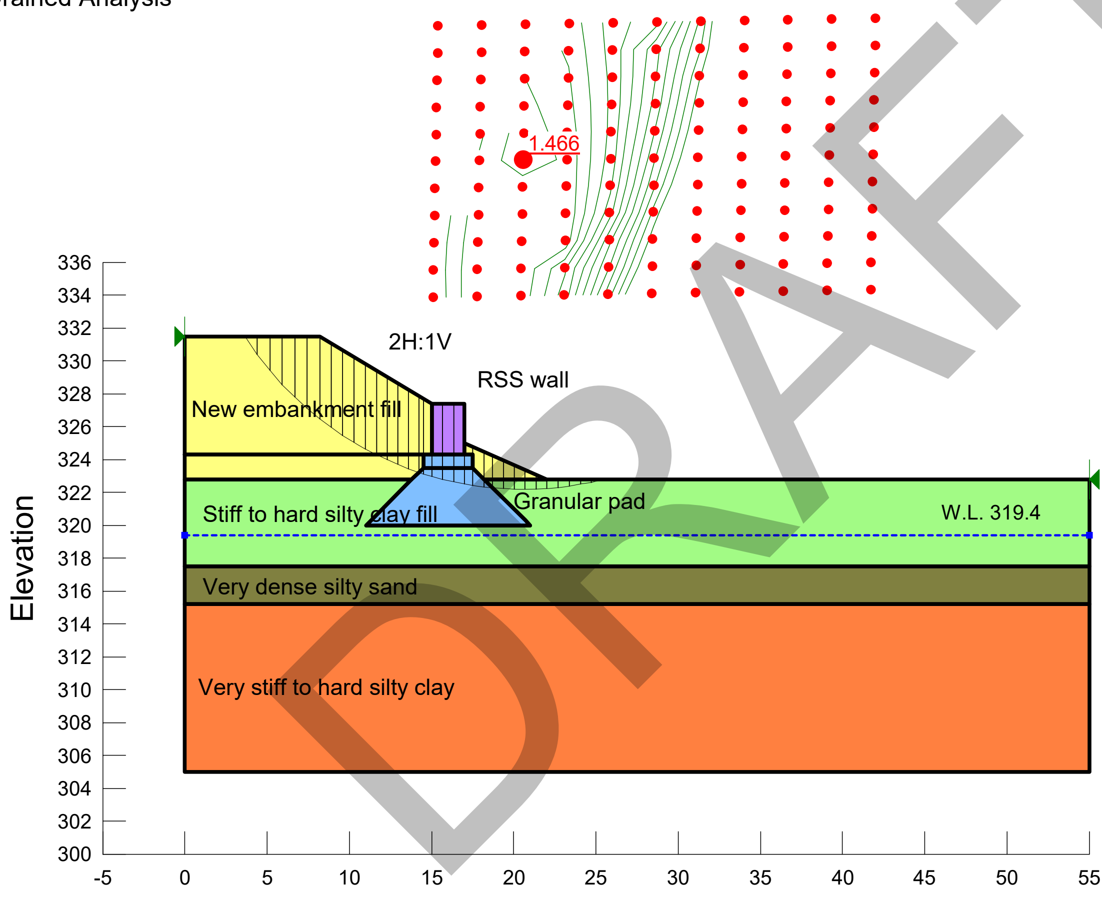

Figure 1F

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Date: 2019-01-21 ,Time: 1:05:01 PM

Name: New embankment Fill Unit Weight: 19 kN/m³ Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1
Name: Stiff to hard silty clay fill Unit Weight: 19 kN/m³ Cohesion: 100 kPa Phi: 0 ° Phi-B: 0 ° Piezometric Line: 1
Name: Very dense silty sand Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 ° Phi-B: 0 ° Piezometric Line: 1
Name: Very stiff to hard silty clay Unit Weight: 20 kN/m³ Cohesion: 200 kPa Phi: 0 ° Phi-B: 0 ° Piezometric Line: 1
Name: RSS Wall Unit Weight: 22 kN/m³ Cohesion: 200 kPa Phi: 45 ° Phi-B: 0 ° Piezometric Line: 1
Name: Granular pad Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 36 ° Phi-B: 0 ° Piezometric Line: 1

Project Number: 11375
Highway 7 - New
N-E Ramp over
S-W ramp, S-W to Wellington St. ramp
and E-N ramp
Retaining wall
Height: 4.5 m approx
Undrained Analysis

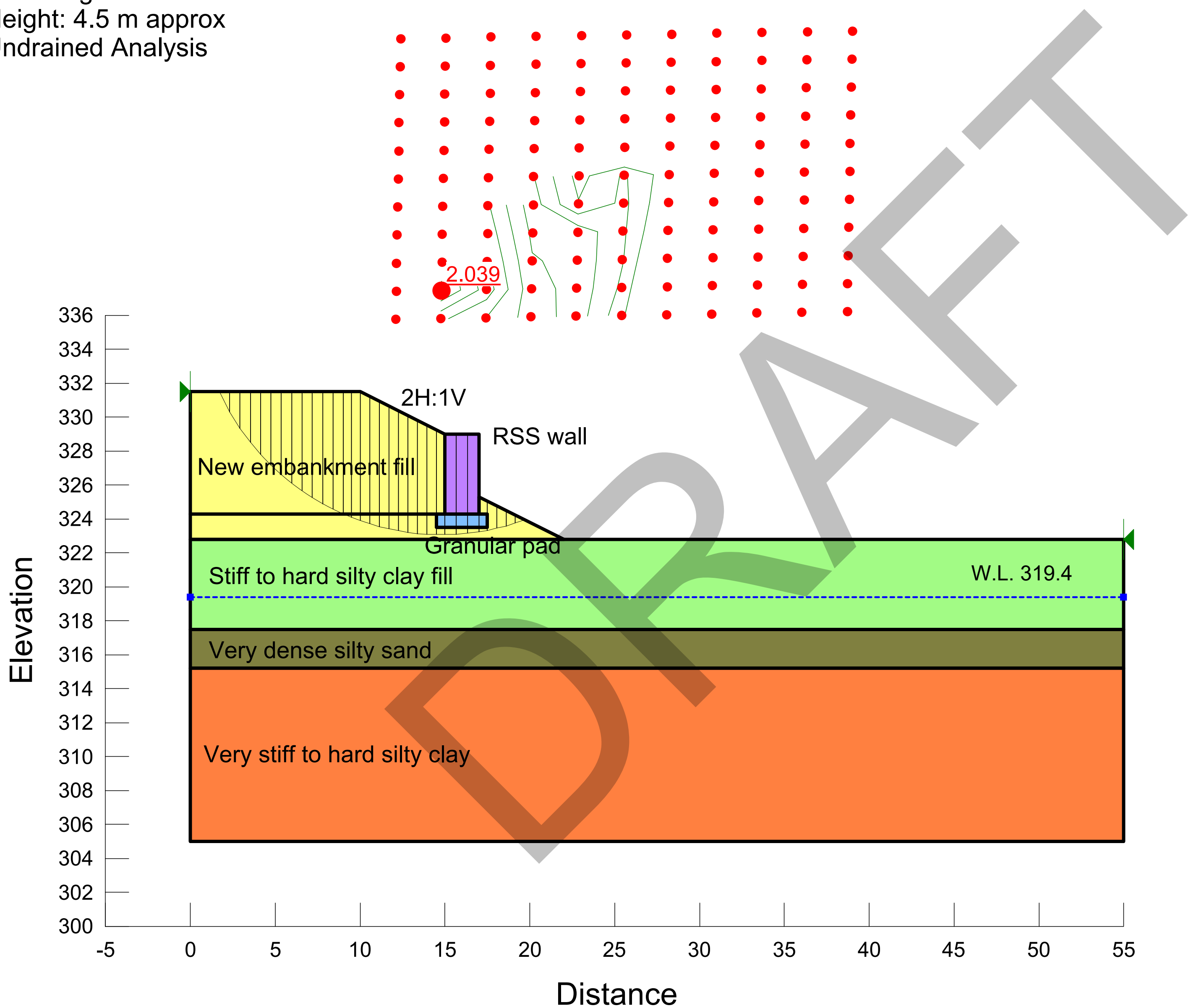

Figure 2F

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Date: 2018-08-28 ,Time: 3:38:45 PM

Name: New embankment Fill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 ° Phi-B: 0 ° Piezometric Line: 1
Name: Hard to firm silty clay fill Unit Weight: 19 kN/m³ Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1
Name: Very dense silty sand Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 ° Phi-B: 0 ° Piezometric Line: 1
Name: Very stiff to hard silty clay Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1
Name: RSS Wall Unit Weight: 22 kN/m³ Cohesion: 200 kPa Phi: 45 ° Phi-B: 0 ° Piezometric Line: 1
Name: Granular pad Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 36 ° Phi-B: 0 ° Piezometric Line: 1

Project Number: 11375
Highway 7 - New
N-E Ramp over
S-W ramp, SW to Wellington St. ramp
and E-N ramp
Retaining wall
Height: 4.5 m approx
Drained Analysis
Seismic Analysis PGA=0.097g

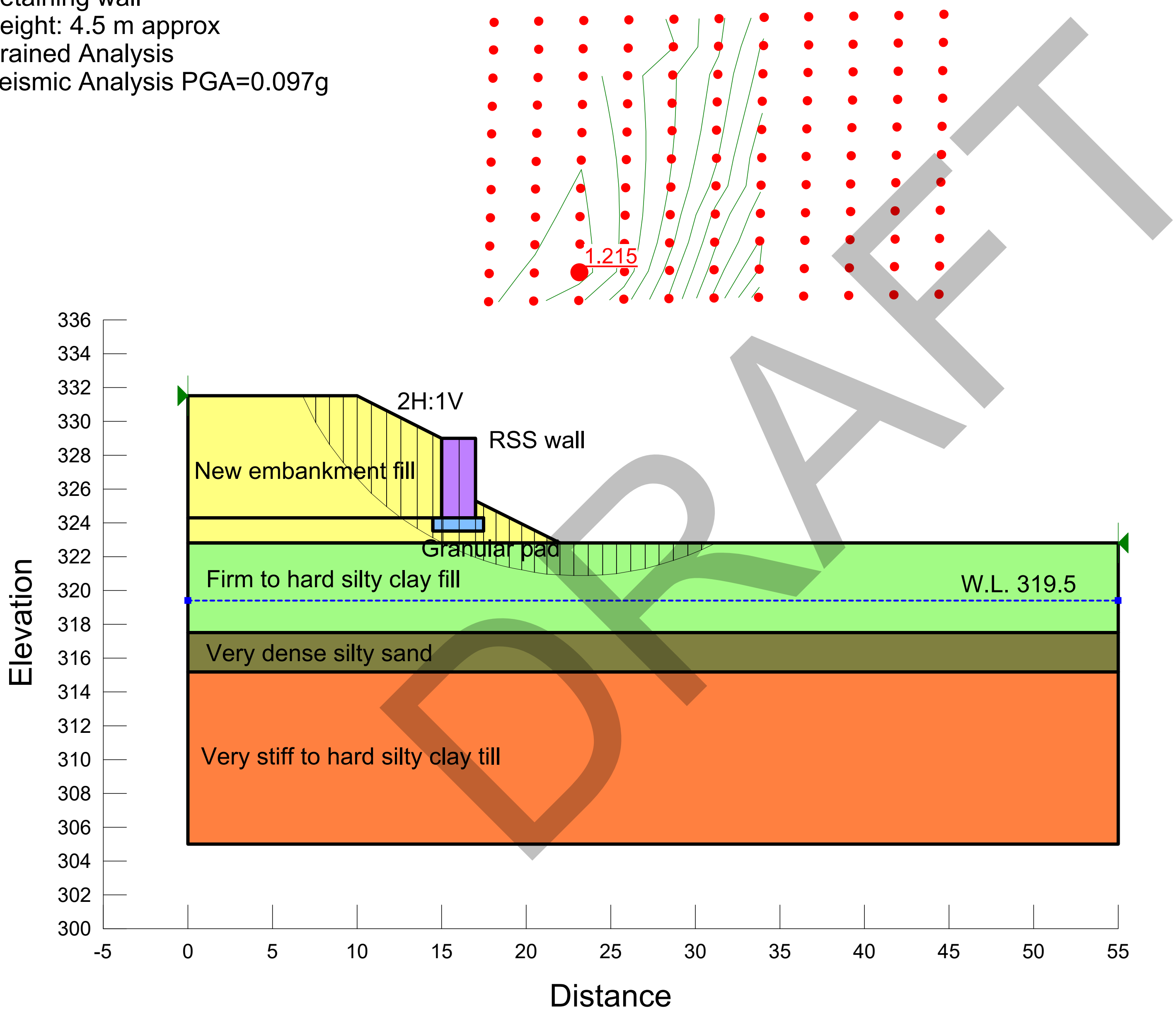

Figure 3F

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Name: New embankment Fill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 ° Phi-B: 0 ° Piezometric Line: 1
Name: Stiff to hard silty clay fill Unit Weight: 19 kN/m³ Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1
Name: Very dense silty sand Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 ° Phi-B: 0 ° Piezometric Line: 1
Name: Very stiff to hard silty clay Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 31 ° Phi-B: 0 ° Piezometric Line: 1

Project Number: 11375
Highway 7 - New
N-E Ramp over
S-W ramp, S-W to Wellington St. ramp
and E-N ramp
Embankment - Side slope
Height: 10 m approx
Drained Analysis

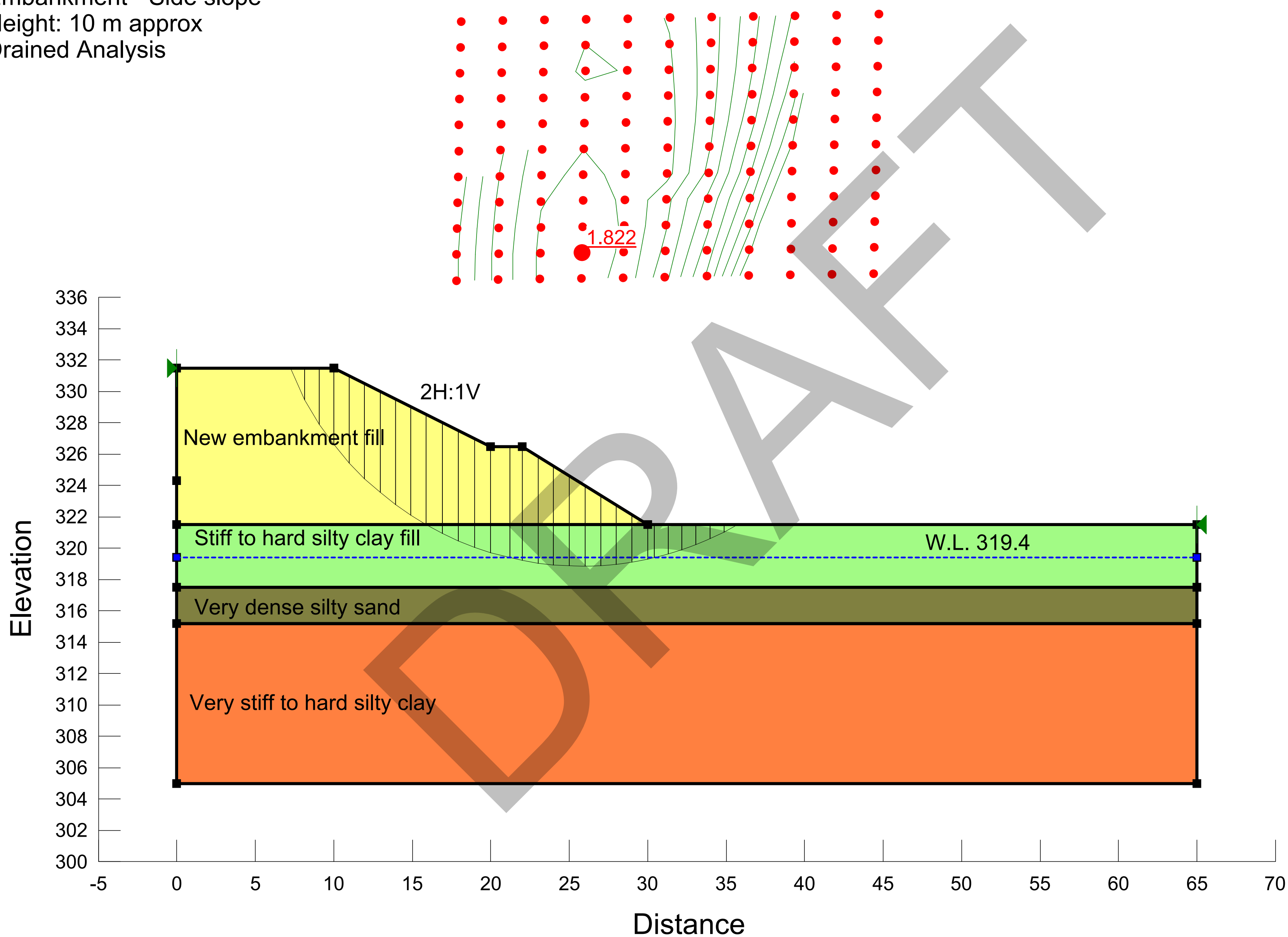

Figure 4F

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Name: New embankment Fill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 ° Phi-B: 0 ° Piezometric Line: 1
Name: Stiff to hard silty clay fill Unit Weight: 19 kN/m³ Cohesion: 100 kPa Phi: 0 ° Phi-B: 0 ° Piezometric Line: 1
Name: Very dense silty sand Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 ° Phi-B: 0 ° Piezometric Line: 1
Name: Very stiff to hard silty clay Unit Weight: 20 kN/m³ Cohesion: 200 kPa Phi: 0 ° Phi-B: 0 ° Piezometric Line: 1

Project Number: 11375
Highway 7 - New
N-E Ramp over
S-W ramp, S-W to Wellington St. ramp
and E-N ramp
Embankment - Side slope
Height: 10 m approx
Undrained Analysis

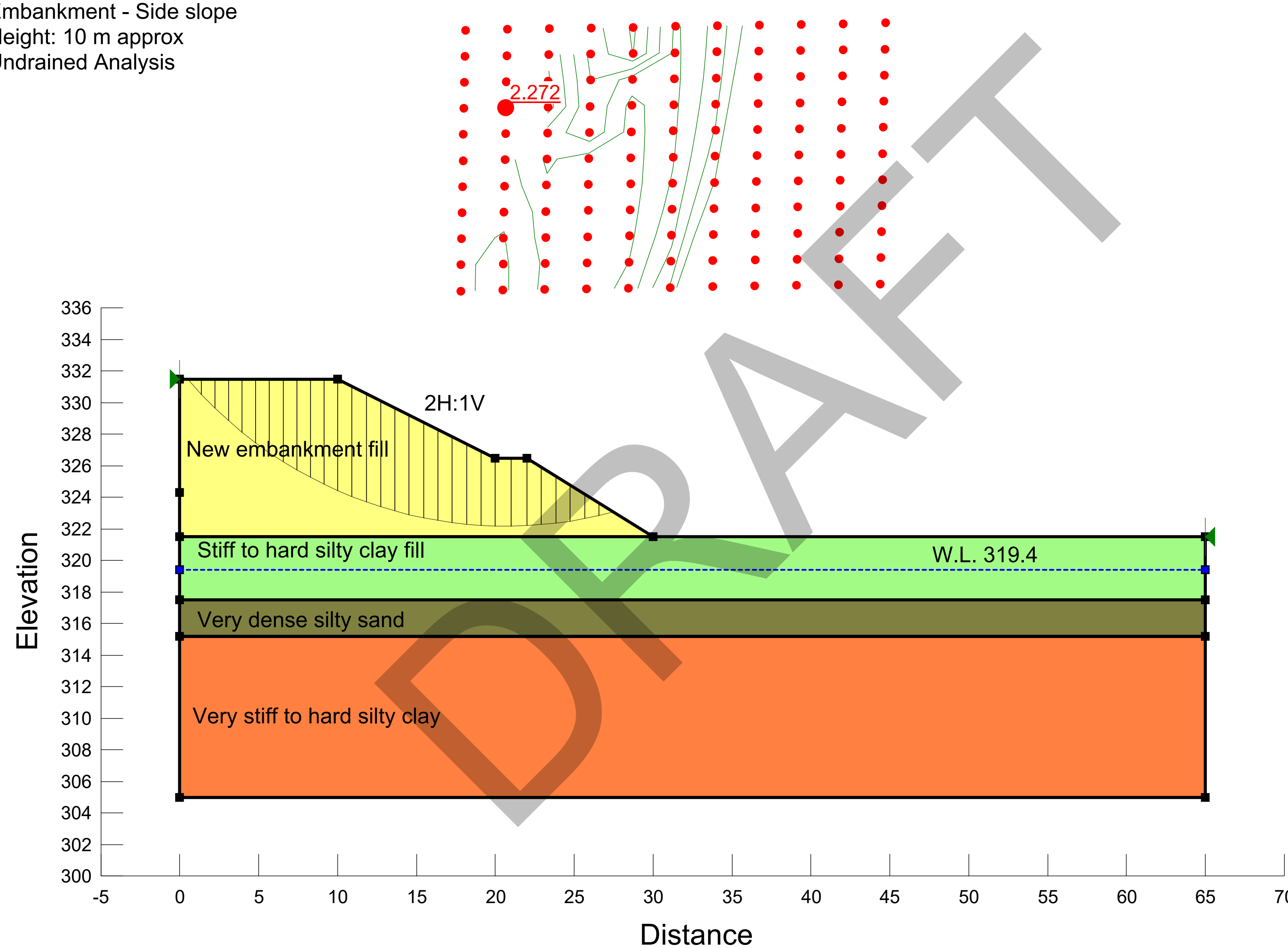

Figure 5F

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Date: 2018-08-28 ,Time: 3:55:09 PM

Name: New embankment Fill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 ° Phi-B: 0 ° Piezometric Line: 1
Name: Stiff to hard silty clay fill Unit Weight: 19 kN/m³ Cohesion: 100 kPa Phi: 0 ° Phi-B: 0 ° Piezometric Line: 1
Name: Very dense silty sand Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 ° Phi-B: 0 ° Piezometric Line: 1
Name: Very stiff to hard silty clay Unit Weight: 20 kN/m³ Cohesion: 200 kPa Phi: 0 ° Phi-B: 0 ° Piezometric Line: 1

Project Number: 11375
Highway 7 - New
N-E Ramp over
S-W ramp, S-W to Wellington St. ramp
and E-N ramp
Embankment - Side slope
Height: 10 m approx
Seismic analysis PGA=0.097g

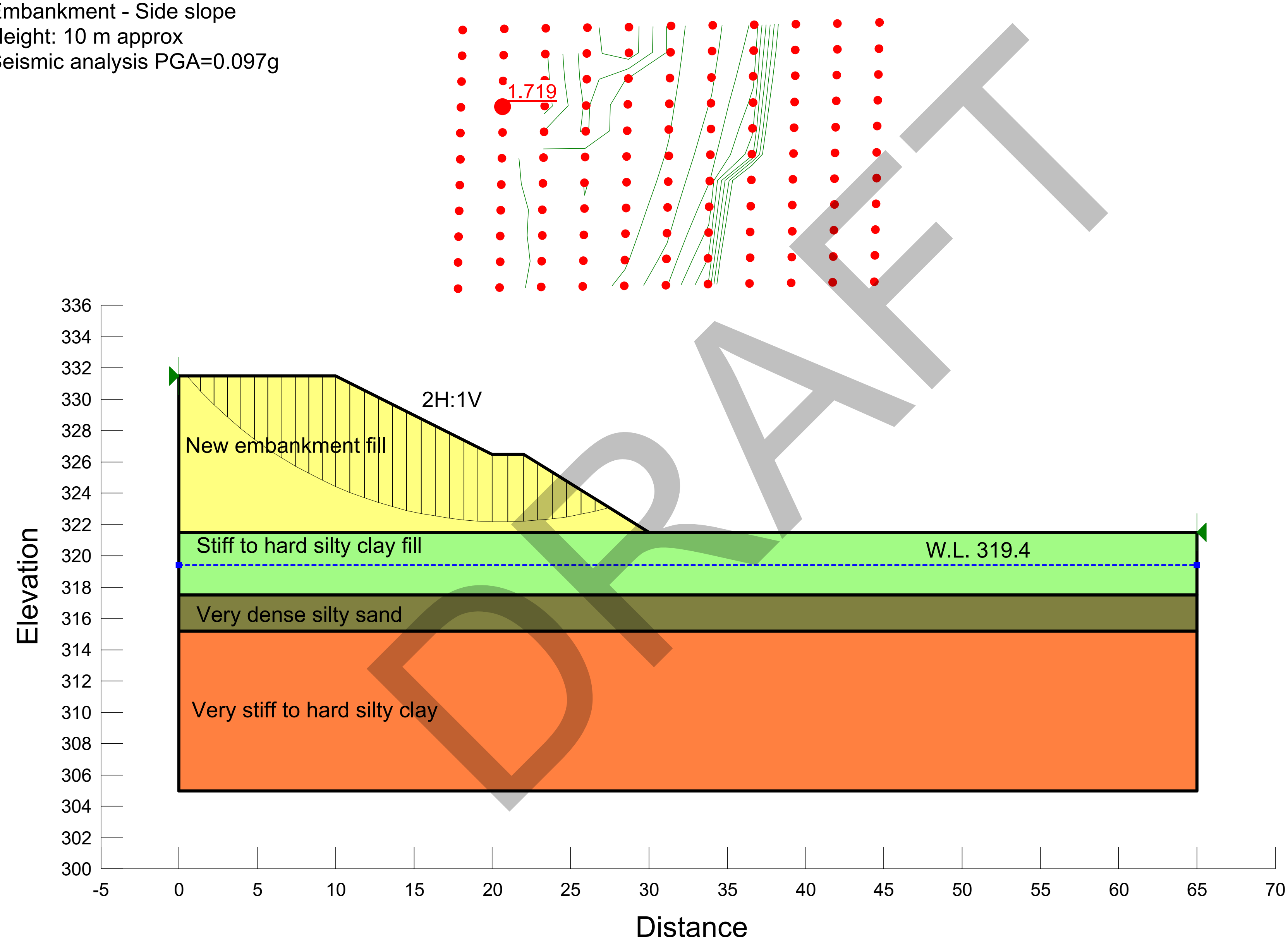

Figure 6F

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Appendix G

List of OPSS Documents and NSSP Wording

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
-	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
-	OPSS PROV 903	Construction specification for deep foundations
-	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 a NSSP on Pile Installation

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

- The cobbles and boulders may impede the driving of the piles resulting in more arduous driving in the very dense soils.
- Some piles may meet refusal on boulders that are large enough not to be dislodged or broken by the pile driving.

- As a result of the presence of boulders, piles may meet refusal at varying depths.
- Pile driving must be controlled according to the criteria specified for the site.

3. 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 for retaining wall construction 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.

4. 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 any adjacent structures or underground utilities:

- The lanes of the Kitchener-Waterloo Express way and Wellington Street will be open to traffic during excavation and foundation construction of NE Ramp over Wellington Street.
- Protection of structure foundations and utilities (if present at this site) during excavation and pile driving.
- Protection of existing approach fills.