



**THURBER** ENGINEERING LTD.

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

**GEOCRES No. 40P9-61**

**Latitude 43.504935 ° , Longitude -80.382280 °**

**Report**

**to**

**WSP**

Date: July 20, 2020  
File: 11375



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SHANTZ STATION ROAD UNDERPASS  
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G.W.P. 408-88-00**

**GEOCRES No. 40P9-61**

**PART 1: FACTUAL INFORMATION**

**1. INTRODUCTION**

This report presents the factual findings obtained from a foundation investigation conducted at the site of the proposed bridge structure to carry Shantz Station Road over Highway 7-New in the Regional Municipality of Waterloo, Ontario. The proposed Shantz Station Road bridge 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 footprints was developed from the data obtained in the course of the investigation.

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

Reference has been made to information on subsurface conditions contained in a previous foundation report prepared for this site during the preliminary design phase. The title of the report is:

- Preliminary, Foundation Investigation and Design Report, Shantz Station Road (RR30) Underpass, Highway 7-New, Kitchener to Guelph, G.W.P. 408-88-00, Geocres No. 40P9-

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47, Report to Ministry of Transportation Ontario Southwestern Region, File: 15-64-17, dated September 2, 2008. (Reference 1).

## **2. SITE DESCRIPTION**

At the site, the Highway 7-New alignment runs approximately parallel to and 350 m north of the existing Highway 7 alignment. The site lies approximately 7.5 km northeast of a developed area of the City of Kitchener and 6.5 km northwest of a developed area of the City of Guelph.

The site is generally flat and lies within an area of active farms and agricultural lands. There are farmsteads to the east and west of Shantz Station Road, near the proposed Highway 7 alignment.

Based on the Ontario Geological Survey Special Volume 2, The Physiography of Southern Ontario, Third Edition by Chapman and Putnam, the site lies within an area referred to as the Guelph Drumlin Field, an area of drumlinized till plain, also mapped as containing eskers. The till is described as stony and the occurrence of surface boulders is noted. Chapman and Putnam give a typical gradation of the till as being 50% sand, 35% silt and 15% clay. Swampy valleys are reported to occur between the drumlins and associated gravel terraces.

## **3. INVESTIGATION PROCEDURES**

A detailed geotechnical investigation was conducted between June 30 and July 27, 2017, and consisted of drilling and sampling five boreholes (numbered SH16-01 to SH16-05) near the foundation units of the proposed underpass. Boreholes SH16-01 and SH16-05 were drilled at the north and south approach embankments respectively. Boreholes SH16-02, SH16-03 and SH16-04 were drilled near the approximate locations of the north abutment, pier and south abutment, respectively. The depths of the boreholes ranged from 6.1 m to 25.0 m (Elevations 323.4 to 305.7). The Record of Borehole sheets of the boreholes drilled during the present investigation, are included in Appendix A.

A preliminary foundation investigation was carried out between May 22 and June 3, 2008. Three boreholes, numbered 08-157, 08-158 and 08-159, were drilled near the north abutment, pier and



south abutment, respectively. The depths of the boreholes ranged from 20.0 m to 21.4 m (Elevations 309.2 to 308.6). The Record of Borehole sheets of the boreholes drilled during the previous investigation, are included in Appendix B.

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

The ground surface elevations and coordinates of the recent as-drilled boreholes were 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 current investigation, track-mounted D52 and truck-mounted CME75 drill rigs were used in conjunction with hollow-stem augers to advance the boreholes in the overburden soils. In general, soil samples were obtained at selected intervals using a 50mm diameter split spoon sampler in conjunction with the Standard Penetration Testing (SPT). NQ coring methods were used to advance Borehole SH16-02 3.1 m into bedrock. Boreholes SH16-03 and SH16-04 were advanced 3.3 m into refusal density soils as defined by SPT 'N' values of greater than 100 blows per 0.3 m of penetration.

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 and rock samples for transport to Thurber's Oakville laboratory for further examination and testing.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers were installed in three selected boreholes (08-158, SH16-02 and SH16-04). Each piezometer consisted of a 25 mm Schedule 40 PVC pipe with a 1.5 m long slotted screen enclosed in a column of filter sand to permit groundwater level monitoring. Piezometer installation details, groundwater level observations and water level readings are



shown on the Record of Borehole sheets. Upon completion of the drilling operations, the boreholes without piezometers were abandoned in general accordance with Ontario Regulation 903 (as amended by O. Reg. 372/07). The details of standpipe piezometer installation and borehole completion are summarized in Table 3.1. The piezometer installations were decommissioned as per O.Reg. 903.

**Table 3.1 – Borehole Completion Details**

Foundation Unit	Borehole Number	Borehole Depth / Base Elevation (m)	Piezometer Tip Depth / Elevation (m)	Completion Details
North Approach	SH16-01	9.8/321.1	None Installed	Backfilled with bentonite and auger cuttings to surface.
North Abutment	08-157	20.8/308.8	None Installed	Backfilled with bentonite benseal grout to 1.5 m, holeplug from 1.5 m to 0.6 m, then auger cuttings to surface.
	SH16-02	25.0/305.7	19.8/310.9	Bentonite holeplug from 25.0 m to 19.8 m, Piezometer with 3 m slotted screen installed with sand filter from 19.8 m to 16.2 m, bentonite holeplug from 16.2 m to 13.2 m, bentonite holeplug and auger cuttings from 13.2 m to 0.15 m, then cement to surface.
Pier	08-158	21.4/308.6	21.4/308.6	Piezometer with 1.5 m slotted screen installed with a sand filter to 19.1 m, holeplug from 19.1 m to 18.3 m, bentonite from 18.3 m to ground surface.
	SH16-03	21.4/309.0	None Installed	Backfilled with bentonite holeplug and auger cutting to surface
South Abutment	08-159	20.0/309.2	None Installed	Grout from 20.0 m to 15.0 m, holeplug from 15.0 m to 3.0 m and auger cuttings from 3.0 m to ground surface.
	SH16-04	21.5/308.5	21.3/308.7	Bentonite holeplug from 21.6 m to 21.3 m, Piezometer with 3 m slotted



Foundation Unit	Borehole Number	Borehole Depth / Base Elevation (m)	Piezometer Tip Depth / Elevation (m)	Completion Details
				screen installed with sand filter from 21.3 m to 17.7 m, bentonite holeplug from 17.7 m to 12.2 m, auger cuttings from 12.2 m to 0.15, then cement to surface.
South Approach	SH16-05	6.1/323.4	None Installed	Backfilled with bentonite holeplug and auger cutting to surface.

#### 4. LABORATORY TESTING

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

#### 5. DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendices A and B. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following paragraphs. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description and



must be used for interpretation of the site conditions. It should be recognized and expected that soil conditions may vary between and beyond borehole locations.

In general, the subsurface conditions encountered at the site consisted of topsoil and/or cohesionless fill overlying native layers of loose to dense silt, loose to very dense gravelly sand, very stiff to hard silty clay till and very dense silty sand till/sandy silt till. Interbedded layers of sand, silty sand and silt were encountered at various depths within the silty clay till. Dolostone bedrock was contacted in one borehole below the cohesionless glacial till. Descriptions of the individual strata are presented below.

### **5.1 Topsoil**

Topsoil was identified at the ground surface in Boreholes 08-157, 08-159, and SH16-03. The topsoil thickness ranged from 75 mm to 200 mm.

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**

Fill was encountered below the topsoil in Boreholes 08-157, and 08-159, and surficially in Boreholes 08-158, SH16-01, SH16-02, SH16-04 and SH16-05. The fill consisted of layers of brown to dark brown gravelly sand, sand, silty sand and silty clay. The thickness of the fill ranged from 0.5 m to 2.2 m, and the depth to the base ranged from 0.5 m to 2.2 m (Elevations 330.2 to 327.4).

SPT 'N' values of the cohesive fill were 4 and 11 blows per 0.3 m of penetration, indicating a firm to stiff consistency. The SPT 'N' values in the cohesionless fill varied from 6 to 48 blows per 0.3 m of penetration, indicating a loose to dense state. The natural moisture content ranged from 4 percent to 21 percent.

### **5.3 Gravelly Sand, Sand and Gravel, and Gravel**

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A layer of brown gravelly sand containing some silt was encountered in Boreholes SH16-01 at 3.0 m depth (Elevations 327.9). The thickness of the gravelly sand layer was 1.4 m. The depth to the base of the gravelly sand was 4.4 m (Elevations 326.4).

Brown sand and gravel was contacted within the silty clay till at 1.4 m depth (Elevation 328.1) in Borehole SH16-05. The thickness of the sand and gravel layer was 0.8 m.

SPT 'N' values in the gravelly sand, and sand and gravel ranged from 4 to 66 blows per 0.3 m of penetration indicating a compact to very dense relative density. . The natural moisture contents in the gravelly sand, sand and gravel, and gravel generally lay in the range of 6 percent to 9 percent.

The grain size distribution curve of a sand and gravel sample tested is presented on the Record of Borehole sheets and on Figure A1 of Appendix A. The result of a laboratory test carried out on a selected sample are as follows:

Soil Particles	(%)
Gravel	39
Sand	37
Silt and Clay	24

#### 5.4 Silt and Sandy Silt

Layers of brown silt and sandy silt containing some sand, trace to some gravel and trace to some clay were contacted at depths ranging from 0.1 m to 0.7 m (Elevations 330.2 to 329.3) in Boreholes SH16-01, SH16-03 and SH16-04. The thickness of the silt and sandy silt varied from 0.7 m to 3.8 m. The depth to the base of the silt and sandy silt ranged from 1.4 m to 5.9 m (Elevations 328.6 to 324.9).

Interbedded layers of silt and silty sand were encountered within the silty clay till/clayey silt till in Boreholes SH16-03, SH16-04, 08-157 and 08-159, at various elevations, generally ranging from 321.4 to 318.3. The thickness of these interbedded layers varied between 0.5 m and 1.6 m.



SPT 'N' values in the silt and sandy silt layers were 2 to 57 blows per 0.3 m of penetration, indicating a very loose to very dense state. The natural moisture contents of the silt and sandy silt generally lay in the range of 7 percent to 21 percent.

Grain size distribution curves of silt samples tested are presented on the Record of Borehole sheets and on Figure A2 and B3 of Appendices A and B, respectively. The results of laboratory tests carried out in the silt samples were as follows:

Soil Particles	(%)
Gravel	0 to 1
Sand	15 to 16
Silt	76 to 80
Clay	4 to 7

### 5.5 Silty Clay, Silty Clay Till and Clayey Silt Till

An extensive deposit of silty clay, silty clay till and clayey silt till was contacted in all the boreholes at depths ranging from 0.5 m to 5.9 m (Elevations 329.5 to 324.9). The cohesive deposit contained trace to some gravel and trace to some silt. Although not specifically identified in the boreholes, glacial tills are known to contain cobbles and boulders. Interbedded layers of sand, silt, silty sand and gravel, were encountered within the silty clay till and clayey silt till and are described in other sections of this report. The thickness of the clayey silt till and silty clay till ranged from 12.1 m to 16.7 m, and locally 4.4 m in Borehole SH16-05 which was terminated at a shallower depth.

A 1.8 m thick layer of grey silty clay was encountered at 8.5 m depth (Elevation 321.1) in Borehole 08-157.

The depth to the base of the cohesive tills ranged from 15.7 m to 20.9 m (Elevations 313.7 to 309.8), and at 5.9 m (Elevation 323.6) in Borehole SH16-05.

Borehole SH16-01 was terminated within the silty clay till at 9.8 m depth (Elevation 321.1).



SPT 'N' values in the silty clay, silty clay till and clayey silt till ranged from 12 to greater than 100 blows per 0.3 m of penetration, indicating a stiff to hard consistency. The natural moisture contents of the silty clay, silty clay till and clayey silt till generally lay in the range of 5 percent to 38 percent.

Grain size distribution curves of silty clay, silty clay till and clayey silt till samples are presented on the Record of Borehole sheets and on Figures A3 and A4 of Appendix A, and Figures B1, B2 and B4 of Appendix B. The results of laboratory tests carried out in the cohesive soil samples were as follows:

Soil Particles	(%)
Gravel	0 to 16
Sand	0 to 34
Silt	29 to 78
Clay	15 to 69

The results of Atterberg Limits are presented on the Record of Borehole sheets and on Figure A7 of Appendix A and Figures B6 to B8 Appendix B. The results of Atterberg Limits testing are summarized below:

Liquid Limit	15 to 55
Plastic Limit	10 to 23

The above results show that the silty clay and clayey silt till/silty clay till are of low to high plasticity with group symbols of CL-ML, CL, CI and CH.

Glacial tills inherently contain cobbles and boulders.

## 5.6 Sand to Silty Sand

A layer of grey sand containing some silt and clay, and trace gravel was encountered in Borehole SH16-02 at a depth of 13.3 m (Elevation 317.4). The thickness of the sand layer was 3.0 m.





A 1.0-m thick layer of grey silty sand containing some gravel, was contacted within the silty clay till in Borehole SH16-04 at 5.9 m depth (Elevation 324.1).

The depth to the base of the sand and silty sand layers was at 16.3 m and 6.9 m (Elevations 314.4 and 323.2), respectively.

SPT 'N' values measured in the sand were 49 and 62 blows per 0.3 m of penetration, indicating a dense to very dense condition. An SPT 'N' value in the silty sand in Borehole SH16-04 was 100 blows per 0.075 m of penetration, indicating a very dense state. The natural moisture contents of the sand and silty sand generally lay in the range of 5 percent to 15 percent.

The grain size distribution curve of a sand sample tested is presented on the Record of Borehole sheets and on Figure A5 of Appendix A. The result of a laboratory test carried out on a selected sample are as follows:

Soil Particles	(%)
Gravel	2
Sand	79
Silt and Clay	19

## 5.7 Sandy Silt Till, Silty Sand Till and Silt Till

Brown to grey sandy silt till and silty sand till containing some gravel to gravelly and trace clay was encountered below the silty clay till/clayey silt till in all the boreholes, except in Borehole SH16-01. The sandy silt till/silty sand till layers were contacted at depths ranging from 15.7 m to 20.9 m (Elevations 313.7 to 309.8). Clayey zones were encountered within the sandy silt till/silty sand till, near Elevations 311.6 to 309.6.

Upper layers of sandy silt till/silty sand till were contacted at 4.1 m and 5.9 m depth (Elevations 325.1 and 323.6) in Boreholes 08-159 and SH16-05.

A 700-mm thick layer of silt till was encountered at 10.2 m depth (Elevation 319.0) in Borehole 08-159.



The depth to the base of the silty sand till was at 21.9 m depth (Elevation 308.7) in Borehole SH16-02. The depth to the base of the sandy silt till was at 5.6 m (Elevation 323.5) in Borehole 08-159.

Boreholes SH16-03, SH16-04, 08-157, 08-158 and 08-159, were terminated within the sandy silt till/silty sand till at depths ranging from 20.0 m 21.5 m (Elevations 309.2 to 308.4). Borehole SH16-05 was terminated within the silty sand till at 6.1 m depth (Elevation 323.5).

SPT 'N' values ranged from 65 blows per 0.3 m of penetration to greater than 100 blows per 0.05 m of penetration indicating a very dense relative density. The natural moisture contents generally lay in the range from 8 percent to 18 percent.

Grain size distribution curves for samples tested are presented on the Record of Borehole sheets and on Figure A6 of Appendix A and Figures B3 and B5 and Appendix B. The results of the laboratory tests carried out on sandy silt till/silty sand till samples are summarized as follows:

Soil Particles	Sand and Silt Till (%)	Silt Till (%)
Gravel	4 to 23	0
Sand	40 to 47	8
Silt	23 to 41	82
Clay	7 to 15	10

Atterberg Limits tests were conducted in the clayey zone of the silty sand till/sandy silt till, and the results are presented on the Record of Borehole sheets and on Figure B9 of Appendix B. The results of Atterberg Limits testing are summarized below:

Liquid Limit	16
Plastic Limit	10

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



Although not specifically identified in the boreholes, this layer may contain cobbles and boulders which may account for some high SPT 'N' values and resistance to augering.

## **5.8 Bedrock**

The overburden soils described above are underlain by dolostone bedrock, which was encountered at a depth of 21.9 m (Elevation 308.7) in Borehole SH16-02, and proven by coring 3.1 m into the rock. The bedrock was grey in colour. Occasional mechanical breaks were noted throughout the bedrock cores. The bedrock is generally described as moderately weathered.

Borehole SH16-02 was terminated within the bedrock at 25.0 m depth (Elevation 305.7).

Total Core Recovery (TCR) in the bedrock ranged from 98% and 100% with Solid Core Recovery (SCR) of 98% and 100%. The Rock Quality Designation (RQD) determined from the recovered cores was 53% to 86%, indicating fair to good rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 1 to 6. Average unconfined compressive strengths (UCS) of the rock were 94 MPa and 138 MPa, indicating the rock is strong to very strong. These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from the boreholes. A summary of the Point Load Test Results are presented in Appendix A.

## **5.9 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 08-158, SH16-02 and SH16-04 to monitor the groundwater level at the site. The piezometers in SH16-02 and SH16-04 were damaged before water level reading could be taken. The groundwater levels measured in the open boreholes and in the standpipe piezometers are summarized in Table 5.1.

**Table 5.1 – Water Level Measurements**

Foundation Unit	Borehole	Date	Water Level (m)		Remark
			Depth	Elevation	
North Approach	SH16-01	June 30, 2017	6.1	324.7	Open borehole
North Abutment	08-157	May 30, 2008	Dry	-	Open borehole
	SH16-02	July 26, 2017	9.1	321.6	Open borehole
		December 18, 2017	-	-	Piezometer was damaged
Pier	08-158	May 28, 2008	6.8	323.2	Piezometer
		June 2, 2008	8.8	321.2	
		July 15, 2008	9.0	321.0	
		August 14, 2008	8.4	321.6	
	SH16-03	July 27, 2017	12.2	318.1	Open borehole
South Abutment	08-159	June 3, 2008	Dry	-	Open borehole
	SH16-04	July 27, 2017	12.2	317.8	Open borehole
		December 18, 2017	-	-	Piezometer was damaged
South Approach	SH16-05	June 30, 2017	Dry	-	Open borehole

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 silty clay till from Borehole SH16-04 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 A.

**Table 6.1 – Analytical Test Results**

Parameter	Units (Soil)	Test Results
		SH16-04 SS 4 Depth 2.3 m
		(Native Silty Clay Till)
Sulphide	%	<0.02
Chloride	µg/g	94
Sulphate	µg/g	15
pH	No unit	9.11
Electrical Conductivity	µS/cm	144
Resistivity	Ohms.cm	6940
Redox Potential	mV	265

## 7. MISCELLANEOUS

Altech Drilling & Investigative Services of Elmira, Ontario supplied a track-mounted D52 drill rig and a truck-mounted CME75 drill rig and conducted the drilling, sampling and in-situ testing operations for the present investigation.

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

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

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

Details of the previous investigation, conducted in 2008, are presented in Reference 1.



Overall supervision of the field program for the present investigation was conducted by Dr. Nancy Berg, P.Eng.. Interpretation of the data and preparation of the current report was carried out by Ms. R. Palomeque Reyna, P.Eng. and Dr. Nancy Berg, P.Eng.

Mr. Jason Lee, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations projects, reviewed the report.

Thurber Engineering Ltd.



Nancy Berg, Ph.D., P.Eng.  
Geotechnical Engineer



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



P.K. Chatterji, Ph.D., P.Eng.  
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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 Shantz Station Road over Highway 7-New EBL and WBL in the Regional Municipality of Waterloo, Ontario.

The General Arrangement (GA) drawing provided by WSP indicates that the new overpass bridge has two spans, 37.5 m and 35.0 m in length, and approximately 19.7 m to 20.7 m in width, supported by two abutments and one pier. Each of the two integral abutments is designed to be supported by a single row of driven steel H-piles, and the centre pier is to be supported on spread footings.

The future Highway 7 mainline grade of both EBL and WBL will be in a cut at approximate base Elevation 327.0 m, which is approximately 3 to 4 m deep below the existing ground surface. The existing ground surface is approximately at Elevation 331.0 m and 330.0 m at the north abutment and south abutment, respectively. The Shantz Road grade within the structure limits will be at approximate Elevation 335.7 m and 334.5 m at the north abutment and south abutment, respectively. Placement of new fill, ranging from 4.5 m to 5 m in height, will be required at this site to construct the south and north approaches respectively. The forward slopes in front of the south and north abutments are proposed to be 2H:1V with crushed stone slope protection.

This foundation investigation and design report, with the interpretation and recommendations, is

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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 discussions 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 and previous investigations.

## **9. STRUCTURE CLASSIFICATION**

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

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

Based on the above classification and Table 6.1 in Section 6.5.2 in the CHBDC (2019), a consequence factor,  $\psi$ , of 1.0 has been used for assessing ULS and SLS factored geotechnical resistances. Should the consequence classification 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 surficial topsoil and cohesionless fill over layers of native gravelly sand and sandy silt to silt, underlain by an extensive deposit of silty clay till/clayey silt till. Interbedded layers of silt, sand, gravel and silty sand were encountered within the cohesive till. The cohesive till was underlain by a layer of silty





sand till/sandy silt till. Bedrock was proved below the silty sand till/sandy silt till in one borehole. The fill was presumably placed during construction of Shantz Road. The groundwater levels measured at this site ranged from 6.1 m to 12.2 m below the ground surface (Elevations 324.7 to 317.8).

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

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

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 construction 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 stiff to 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	Highest Founding Elevation (m)	Founding Stratum	Factored $ULS_f$ (kPa)	SLS (up to 25 mm settlement) (kPa)
North Abutment	08-157 SH16-02	327.4	Hard silty clay till	450	300
Pier	08-158 SH16-03	327.5	Hard silty clay till	450	300
South Abutment	08-159 SH16-04	327.8	Hard silty clay till	450	300

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

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

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 hard silty clay till may be computed based on an ultimate coefficient of friction,  $\tan \delta$ , of 0.45. A resistance Factor of 0.6 should be applied for cohesive soils and, 0.8 for cohesionless soils, as indicated in Table 6.2 in the CHBDC (2019).

If temporary excavations required to construct these footings extend below the water table (Elev. 325.0 m), local groundwater control and dewatering will be required to construct the footings in the dry and to prevent disturbance of the footing base.



The bases of the foundation excavations should be inspected by a Foundation Specialist to confirm that the exposed subgrade surface conforms to the design requirements and has been adequately prepared to receive concrete. Once approved, the subgrade should be protected by a working mat with a minimum thickness of 100 mm and consisting of mass 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 mass 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 option is selected, all topsoil or other deleterious materials must be stripped from the footprint of the foundation to expose competent native undisturbed subgrade material. Subexcavation of existing surficial fill soils will be required. The engineered fill will bear on native very stiff to 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**

North Abutment (BH 08-157, SH16-02)	Pier (BH 08-158, SH16-03)	South Abutment (BH 08-159, SH16-04)
327.4*	328.0*	327.8*

**Note (\*) Elevations are higher than proposed Highway 7 finish grade (327.0 m)**

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

Factored Geotechnical Resistance at ULS 900 kPa

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 (2019) Clause 6.10.2 to Clause 6.10.5.

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

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

If temporary excavations required to construct the engineered fill pad extend below the water table (Elev. 325.0 m), local groundwater control and dewatering 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 not to 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,  $\tan \delta$ , of 0.55. Resistance Factor of 0.8 should be applied for cohesionless soils, as indicated in Table 6.2 in the CHBDC (2019).

The bases of the foundation excavations should be inspected by a Foundation Specialist to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to place the engineered fill. The Granular A for the engineered fill pad must be compacted to 100% Standard Proctor maximum dry density (SPMDD) at optimum moisture content of  $\pm 2\%$  and placed in 300 mm lifts. The geometry of the fill pad must conform to the general requirements shown in Figure 1 in Appendix 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 potential caisson installation difficulties including basal boiling and heave within the water bearing sand and silt till deposit below the silty clay till. Therefore this option is not recommended and has not been developed further.

### **10.4 Steel H-Piles**

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

The subsurface conditions at the north abutment are considered suitable for the design of foundations supported on piles driven to achieve refusal on bedrock. At the pier and south abutment, the piles should be driven to achieve resistance in the very dense sandy silt till/silty sand till encountered at this site.

The GA drawing indicates that the underside elevation of the abutment stem at the north abutment is 330.0 and at the south abutment is 329.8. If driven piles are required at the pier, the underside of the pile cap is approximately at Elevation 324.0.

#### **10.4.1 Axial Resistance**

The axial resistances of HP 310 X 110 and HP 360 x 132 steel piles, and 324 mm x 12.7 mm and 356 mm x 12.7 mm pipe steel piles driven to refusal in very dense sand and silt till and bedrock were assessed based on the subsurface conditions encountered at the north and south abutments 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 Axial Geotechnical Resistance and 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 Geotechnical Resistance ULS (kN)	Geotechnical Resistance SLS (kN)	Geotechnical Resistance Factored ULS (kN)	Geotechnical Resistance Factored SLS (kN)
North Abutment	08-157 SH16-02	308.7 <sup>(1)</sup>	21.0	2,500	Does not govern	3,000	Does not govern
Pier	08-158 SH16-03	309.0	15.0	1,600	1,400	1,800	1,600
South Abutment	08-159 SH16-04	309.0	21.0	1,600	1,400	1,800	1,600

<sup>(1)</sup> Top of bedrock

**Table 10.4 – Estimated Axial Geotechnical 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 Wall Thickness 12.7 mm		Pile Section 356 mm diameter Wall Thickness 12.7 mm	
				Factored Geotechnical Resistance ULS (kN)	Geotechnical Resistance SLS (kN)	Geotechnical Resistance Factored ULS (kN)	Geotechnical Resistance Factored SLS (kN)
North Abutment	08-157 SH16-02	308.7 <sup>(1)</sup>	21.0	2,100	Does not govern	2,400	Does not govern
Pier	08-158 SH16-03	309.0	15.0	1,300	1,100	1,600	1,400
South Abutment	08-159 SH16-04	309.0	21.0	1,300	1,100	1,600	1,400

<sup>(1)</sup> Top of bedrock

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

For piles driven to bedrock, the axial geotechnical resistances based on the bedrock strength are expected to exceed the factored structural capacity of the piles. Accordingly, the structural capacity of the various pile types indicated above will govern the design and should be used for design. The SLS condition will not govern for piles founded on the bedrock.

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:

Clayey Silt Till/Silty Clay Till (cohesive soils)

$$k_s = 67 C_u / B \quad (\text{kN/m}^3)$$

$$p_{ult} = 9 C_u \quad (\text{kPa}) \text{ at and below a depth of } 3B \text{ reduced to zero at the ground surface}$$

where  $p_{ult}$  = ultimate lateral resistance mobilized by a pile, kPa

$C_u$  = undrained shear strength of cohesive soils, kPa

$\gamma$  = unit weight of soil, kN/m<sup>3</sup>

$B$  = width of pile, m



Sandy Silt Till/Silty Sand Till (cohesionless soils)

$$k_s = n_h \cdot z / B \quad (\text{kN/m}^3)$$

$$p_{ult} = 3 \cdot \gamma' \cdot z \cdot K_p \quad (\text{kPa})$$

where  $z$  = depth of embedment of pile, m

$B$  = pile width, m

$n_h$  = coefficient related to soil density,  $\text{kN/m}^3$ , Table 10.5

$\gamma'$  = Bouyant unit weight of soil,  $\text{kN/m}^3$ , 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 spring constant,  $K$ , for analysis may be obtained by the expression,  $K = k_s \times d_z \times B$  ( $\text{kN/m}$ ), where  $k_s$  is the coefficient of horizontal subgrade reaction ( $\text{kN/m}^3$ ),  $B$  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 the 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)	Undrain ed Shear Strengt h $C_u$ (kPa)	Unit Weight $\gamma$ ( $\text{kN/m}^3$ )	$K_p$	$n_h$ ( $\text{kN/m}^3$ )	Soil Conditions
	08-157 SH16-02	330.0 to 327.5	-	21	2.9	1,500	Loose to compact fill



North Abutment		327.5 to 317.4	175	10*	-	-	Very stiff to hard silty clay till
		317.4 to 314.4	-	11*	3.8	7,000	Dense to very dense sand
		314.4 to 310.0	125	10*	-	-	Very stiff silty clay till
		310.0 to 306.5	-	11*	4.0	10,000	Very dense sandy silt till/silty sand till
Pier	08-158 SH16-03	330.0 to 328.0	-	20	2.8	1,500	Very loose to loose silt
		328.0 to 325.0	125	20	-	-	Stiff to hard silty clay till
		325.0 to 321.0	150	10*	-	-	Stiff to hard silty clay till
		321.0 to 312.0	125	10*	-	-	Stiff to very stiff silty clay till
		312.0 to 308.8	-	11*	4.0	10,000	Very dense silty sand till/silty sand till
South Abutment	08-159 SH16-04	329.0 to 327.8	75	20	-	-	Stiff silty clay fill
		327.8 to 325.0	150	20	-	-	Very stiff to hard silty clay till
		325.0 to 323.5	-	11*	4.0	10,000	Very dense sandy silt till
		323.5 to 313.5	175	10*	-	-	Very stiff to hard silty clay till
		313.5 to 308.4	-	11*	4.0	10,000	Very dense sandy silt till/silty sand till

\* Buoyant unit weight below the water table



The lateral resistance within the frost depth of 1.4 m should be ignored. The group efficiency factors can be calculated based on side-by-side and line-by-line factors shown in Figures C6.22, C6.23 and C6.24 of the CHBDC (2019), S6:19 (Commentary).

#### **10.4.4 Pile Installation**

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

At the south abutment and pier, pile driving must be controlled in accordance with Standard Provision SS103-11 (Hiley Formula), and an ultimate pile resistance must be specified by the designer. The Hiley formula does not need to be used until the pile tip is within 2 m of the design tip elevation. The appropriate pile driving note to be shown on the contract drawing is “Piles to be driven in accordance with Standard SS103-11 using an ultimate geotechnical resistance of R kN per pile” where “R” must have a minimum value of twice the factored design load at ULS. It is recommended that Pile Driving Analysis (PDA) testing be conducted in conjunction with the Hiley tests at this site, to ensure the integrity of the pile and to verify pile ultimate geotechnical resistance. PDA testing should be completed for 10 percent the piles for each foundation element or a minimum of 2 piles tested at each foundation element, whichever is more.

At the north abutment, the piles must be driven to bedrock. The appropriate pile driving note is “Piles to be driven to bedrock”.

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

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

Appropriate pile tip protection should be provided for open ended pipe piles, if selected.

An NSSP for piles driven to bedrock must be included in the tender documents. 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.

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

### **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 current information, it is recommended that the north abutment be supported on steel H-piles driven to bedrock, and that the south abutment be supported on steel H-piles driven to very dense sandy silt till/sand and silt till. It is recommended that the pier be supported on spread footings founded on native undisturbed very stiff to hard silty clay till.

## **11. RETAINING WALLS**

The GA drawing proposes wingwalls (RSS walls) on both ends of the north abutment. The proposed wingwalls extend out from the abutments and are approximately 5.7 m long.

If RSS walls are used in conjunction with the new bridge abutments, they should be specified to be "High Performance" and "High Appearance". Therefore, it is important that the RSS walls be founded on soils capable of supporting the imposed loading and limiting settlements to within



acceptable magnitudes. Reference should be made to CHBDC (2019) Clause 6.19 for design of the RSS walls.

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.

The performance of a RSS is dependent on, among other factors, the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, failure of the system. It is critical that the RSS walls are not subject to excessive settlement due to compression of the foundation soils. The foundation of the entire RSS mass must be considered, i.e. from the face of the wall to the furthest extent of the reinforcement.

To provide an acceptable foundation performance, the RSS should be founded on a minimum 500 mm thick OPSS Granular A engineered fill pad resting on the native very stiff silty clay till. The engineered fill pad should extend a minimum 500 mm beyond the footprint of the RSS mass. The design of RSS wall bearing on native undisturbed soil at or below elevations indicated in Table 11.1 should be designed using a factored geotechnical resistance shown in Table 11.1.

**Table 11.1 – Geotechnical Resistance and Founding Elevation of Engineered Fill Pad**

Foundation Unit	Borehole	Depth below existing ground surface (m)	Approx. Founding Elevation (m)	Founding Stratum	Geotechnical Resistance (kPa)	
					Factored ULS (kPa)	Factored SLS (kPa)
North Abutment	08-157 SH16-02	2.6 to 3.7	327.0	Dense silt/Very stiff silty clay till	350	250



The geotechnical resistances provided above are for concentric, vertical loading. The effects of load inclination and eccentricity need to be taken into account according to the CHBDC (2019) Clauses 6.10.2 to 6.10.5.

As per MTO RSS Design Guidelines, the minimum soil cover to the underside of the levelling pad shall be at least 800mm, or 40% of the actual frost depth for the area, whichever is greater.

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 native very stiff silty clay till and dense silt may be estimated using an ultimate friction coefficient of 0.45 and 0.5, respectively. As per Table 6.2 in CHBDC 2019, a resistance factor of 0.6 for cohesive soils and 0.8 for cohesionless soils should be applied to the above value.

Topsoil, organics, loose fill, disturbed soils and deleterious 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 described 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 and be selected from the MTO DSM List for RSS.

### 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 3.5 m high wall founded on very stiff silty clay till, with a back slope of 2H:1V.

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 0.094g. 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)
<b>RSS wall up to 3.5 m high</b>		
Drained	1.6	1F
Undrained	1.6	2F
Seismic = 0.094g	1.3	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. Under the assumed seismic loading, the minimum acceptable factor of safety is 1.1. Accordingly, the computed factors of safety are considered to be acceptable for the proposed RSS wall configuration.

### 11.2 Settlement of the Retained soil system

The construction of a maximum 3.5 m high RSS wall on a 0.5 m thick pad of granular engineered fill will induce immediate (elastic) settlement in the underlying compact/very stiff to hard cohesionless soils



The immediate settlements were assessed using elastic methods. Based on these analyses, the settlement is estimated to be less than 25 mm.

This settlement will be essentially complete when construction of the RSS wall 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 2019 but are generally given by the expression:

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

where:

$p_h$	=	horizontal pressure on the wall at depth $h$ (kPa)
$K$	=	earth pressure coefficient (see Table 12.1)
$\gamma$	=	unit weight of retained soil (see Table 12.1)
$h$	=	depth below top of fill where pressure is computed (m)
$q$	=	value of any surcharge (kPa).

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

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

**Table 12.1 – Earth Pressure Coefficients**

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

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

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

The factors in Table 12.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to be used in the design can be estimated from Figure C6.27 in the Commentary to the CHBDC 2019.

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





### 13. APPROACH EMBANKMENTS

Based on the GA drawing, the proposed finished grade at the structure will be at about Elevations 335.7 and 334.5 at the north and south abutments, respectively. The finished grade level of the Highway 7 New will be at Elevations 327.0 m. The existing ground surface is approximately at Elevation 331.0 m and 330.0 m at the north abutment and south abutment, respectively. Earth cut of about 3 m to 4 m will be required to achieve the Highway 7 New final grade. Placement of new fill, ranging from 4.5 m to 5 m in height, will be required at this site to construct the south and north approaches respectively.

All embankment fill must be constructed with adequate quality control in accordance with OPSS.PROV 206 and OPSS.PROV 501 requirements and the clean earth fill must not contain medium or high plastic clay.

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

Where the total height of the approaches embankments (cut and fill) 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

It is estimated that at the approach embankments, immediate settlements in the order of less than 25 mm will occur in the foundation soils under the loading imposed by approximately up to 5 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.



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

#### **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 and SP 109S12.

Excavation for foundation construction will be extended through the cohesionless fill, and into the native silt, gravelly sand, silty clay till and clayey silt till.

For the purposes of the OHSA, the fill and native soils above the water table are classified as Type 3, and the glacial till above the water table may be classified as Type 2. 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 clean earth fill may be used.



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

Permanent earth cuts are required to construct the Highway 7 New EBL and SBL and the immediate approaches of Shantz Road bridge at this site. The earth cut will be formed through about 4 m to 5 m of soils, typically consisting of very stiff silty clay fill, compact gravelly sand fill, compact sandy silt. It is anticipated that the base of the cut will consist of silty clay till and silt. The groundwater levels measured at this site ranged from 6.1 m to 12.2 m below the ground surface (Elevations 324.7 to 317.8).

At the maximum depth, the cut slopes are expected to be stable at inclinations not steeper than 2H:1V.

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

- Storm runoff
- Seepage from the sides of the cut and cohesionless soils.

It is recommended that the seepage be controlled by means of permanent drains incorporated within the abutment design.

Temporary drainage of the cuts should be provided to maintain a relatively dry, stable excavation. Positive drainage of the permanent cuts must be provided. Roadside ditches are expected to provide an adequate level of surface drainage in most areas where applicable. An interceptor ditch should be provided at the top of the earth cuts as per OPSD 200.020 and 201.020.

The vegetative cover should be established on all exposed earth cut slopes to protect against surficial erosion. Reference may be made to OPSS.PROV 804. Localized sloughing may occur



in cut slopes, particularly in the few years following construction and until the vegetation cover is well established.

## 17. GROUNDWATER AND SURFACE WATER CONTROL

The groundwater levels measured at this site ranged from 6.1 m to 12.2 m below the ground surface (Elevations 324.7 to 317.8). Seasonal fluctuations of the groundwater level are to be expected.

In general, seepage from perched water from the cohesionless fill is to be expected. For temporary foundation excavations at this site, groundwater control will likely be limited to diverting surface runoff and preventing precipitation from entering the excavations and supplemented by sump pumping within the foundation excavation. Filtered sumps must be properly designed to control loss of fines/ground loss. Dewatering must remain operational and effective until the footings are backfilled.

Temporary excavation of the cohesionless native soils below the groundwater level without prior dewatering is not recommended since the inflow of groundwater will cause boiling and sloughing of the soil below the water table making it difficult to maintain a dry, sound base on which to work.

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 and gravel	$4.2 \times 10^{-3}$
Silt	$5.6 \times 10^{-5}$
Silty clay till	$8.0 \times 10^{-7}$
Sand	$5.6 \times 10^{-3}$
Silty sand/sandy silt till	$2.5 \times 10^{-5}$



Dewatering of all excavations should be carried out in accordance with OPSS. PROV 517, SP 517F01 Amendment to OPSS 517, November 2016 (issued July 2017), and OPSS. PROV 902 and SP 109S12.

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.

## 18. ROADWAY PROTECTION

If roadway protection is required during construction of the proposed underpass, 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:

$\gamma$	=	20 kN/m <sup>3</sup>
$\gamma_w$	=	10 kN/m <sup>3</sup>
$K_a$	=	0.33 (fills)
	=	0.36 (silt)
	=	0.35 (gravelly sand)
	=	0.31 (silty clay till)
$K_p$	=	3.0 (fills)
	=	2.8 (silt)



= 2.9 (gravelly sand)  
= 3.2 (silty clay till)

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.

## 19. SEISMIC CONSIDERATIONS

In accordance with the CHBDC 2019, the selection of the seismic site classification is based on the averaged soil conditions encountered in the upper 30 m of the stratigraphy. The stratigraphy of the site includes which consists of loose to dense fill overlying loose to dense silt, very stiff to hard clayey silt till to silty clay till and very dense sandy silt till/silty sand till. This would correspond to a Seismic Site Class D in accordance with Table 4.1, Clause 4.4.3.2 of the CHBDC. The peak ground acceleration, PGA, for a 2% in 50-year probability of exceedance at this site is 0.076 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 0.094 g.

In accordance with Clause 6.14.7 of the CHBDC 2019, retaining structures should be designed using active ( $K_{AE}$ ) and passive ( $K_{PE}$ ) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 19.1 may be used:

**Table 19.1 – Earth Pressure Coefficients for Earthquake Loading**

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

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

\*\* After Woods

Based on review of the subsurface information and SPT data, seismically induced liquefaction of foundation soils is not considered to be a concern at this site.

## 20. CORROSION AND SULPHATE ATTACK POTENTIAL

The results of the corrosivity and sulphate analytical tests conducted on the native silty clay till soil during the current investigation indicates the following conditions at the location tested:

- The potential for sulphate attack on concrete foundations from the surrounding native silty clay soil is considered to be negligible due to the low concentration of sulphate and chloride in the sample 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 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. ADJACENT 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.

## **22. CONSTRUCTION CONCERNS**

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

### **1. Pile Installation**

Bedrock was proved by coring only in Borehole SH16-02, drilled at the north abutment. Since the elevation of the bedrock surface was only established at one location, it is possible that higher or lower bedrock elevations will be encountered during construction/pile installation. Therefore, the lengths of piles driven to bedrock might vary.

### **2. Excavation**

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

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

### **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



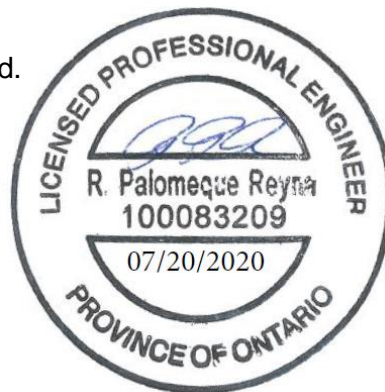
measures must be in place prior to commencing excavation. All footings must be constructed in the dry.

## 23. CLOSURE

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

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

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Review Principal, Designated MTO Contact



## **Appendix A**

### **Record of Borehole Sheets, Laboratory Test Results, and Analytical Laboratory Test Results (Current Investigation)**

## SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

### 1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

### 3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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



### 4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

### 5. LEGEND FOR RECORDS OF BOREHOLES

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

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


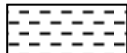



 Water Level  
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value      Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT      Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

# UNIFIED SOILS CLASSIFICATION

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

## EXPLANATION OF ROCK LOGGING TERMS

<u>ROCK WEATHERING CLASSIFICATION</u>		<u>SYMBOLS</u>	
<b>Fresh (FR)</b>	No visible signs of weathering.		
<b>Fresh Jointed (FJ)</b>	Weathering limited to the surface of major discontinuities.		CLAYSTONE
<b>Slightly Weathered (SW)</b>	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
<b>Moderately Weathered (MW)</b>	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
<b>Highly Weathered (HW)</b>	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
<b>Completely Weathered (CW)</b>	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Very thinly bedded	20 to 60mm				
Laminated	6 to 20mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Thinly Laminated	Less than 6mm				

<u>TERMS</u>		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.				
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

# RECORD OF BOREHOLE No SH16-01

1 OF 2

METRIC

GWP# 408-88-00 LOCATION Shantz Station Road, MTM NAD 83 Zone 10: N 4 818 763.6 E 233 428.9 ORIGINATED BY GA  
DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA  
DATUM Geodetic DATE 2017.06.30 - 2017.06.30 LATITUDE 43.505385 LONGITUDE -80.382633 CHECKED BY JPL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE						PLASTIC LIMIT  w <sub>P</sub> NATURAL MOISTURE CONTENT  w LIQUID LIMIT  w <sub>L</sub> WATER CONTENT (%)				
330.8	GROUND SURFACE							20	40	60	80	100						
0.0	Gravelly SAND Dense Grey Moist (FILL)		1	SS	48													
330.2																		
0.7	SILT, some sand, trace clay, trace gravel Loose to Dense Brown Moist		2	SS	5		330											
			3	SS	9		329											
			4	SS	32		328											
327.9																		
3.0	Gravelly SAND, some silt Very Dense Brown Moist		5	SS	66		327											
326.4																		
4.4	Dense Wet		6	SS	45		326										0 15 79 6	
324.9							325											
5.9	Silty CLAY, some sand to sandy, trace gravel Hard Grey Moist (TILL)		7	SS	66												3 25 47 25	
							324											
	Wet		8	SS	36		323											
							322											
			9	SS	32													
321.1																		
9.8	END OF BOREHOLE AT 9.8m.																	

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-11375(GINTDATA)\GPJ 2017TEMPLATE(MTO).GDT 6/1/20

# RECORD OF BOREHOLE No SH16-01

2 OF 2

METRIC

GWP# 408-88-00 LOCATION Shantz Station Road, MTM NAD 83 Zone 10: N 4 818 763.6 E 233 428.9 ORIGINATED BY GA  
 DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA  
 DATUM Geodetic DATE 2017.06.30 - 2017.06.30 LATITUDE 43.505385 LONGITUDE -80.382633 CHECKED BY JPL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
	Continued From Previous Page																
	WATER LEVEL AT 6.1m. BOREHOLE BACKFILLED WITH BENTONITE AND AUGER CUTTINGS TO SURFACE.																

ONTMT4S2 MTO-11375(GINTDATA)\GPJ 2017TEMPLATE(MTO).GDT 6/1/20

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
5  
0  
 (%) STRAIN AT FAILURE

## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

ONTMT4S2 MTO-11375(GINTDATA).GPJ 2017TEMPLATE(MTO).GDT 6/4/20



# RECORD OF BOREHOLE No SH16-02

2 OF 3

METRIC

GWP# 408-88-00 LOCATION Shantz Station Road, MTM NAD 83 Zone 10: N 4 818 751.2 E 233 451.1 ORIGINATED BY OA  
 DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY MFA  
 DATUM Geodetic DATE 2017.07.25 - 2017.07.26 LATITUDE 43.505276 LONGITUDE -80.382356 CHECKED BY JPL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
	Continued From Previous Page															
317.4	Silty <b>CLAY</b> , sandy, trace gravel Very Stiff to Hard Brown to Grey (TILL)		10	SS	100/ .175		320									
							319									
			11	SS	23		318									
13.3	<b>SAND</b> , some silt, trace clay, trace gravel Dense to Very Dense Grey Wet		12	SS	49		317									
							316									
			13	SS	62		315									
314.4	Silty <b>CLAY</b> , sandy, trace gravel Very Stiff to Hard Grey (TILL)		14	SS	22		314									
16.3							313									
			15	SS	23		312									
							311									
			16	SS	100/											

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

ONTMT4S2 MTO-11375(GINTDATA)GPJ 2017TEMPLATE(MTO).GDT 6/4/20

## METRIC

30" x 30" x 5'	SAMPLES			DYNAMIC CONE PENETRATION			
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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No SH16-03

1 OF 3

METRIC

GWP# 408-88-00 LOCATION Shantz Station Road, MTM NAD 83 Zone 10: N 4 818 710.2 E 233 448.4 ORIGINATED BY GA  
 DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA  
 DATUM Geodetic DATE 2017.07.27 - 2017.07.27 LATITUDE 43.504907 LONGITUDE -80.382385 CHECKED BY JPL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE							WATER CONTENT (%) W <sub>P</sub> W      W <sub>L</sub>			
330.3	GROUND SURFACE							20	40	60	80	100		20	40	60		
0.0	TOPSOIL: (100mm)																	
0.1	SILT, some clay, some gravel, trace sand, occasional organics Loose to Very Loose Brown Moist		1	SS	7		330							○				
			2	SS	3		329							○				
			3	SS	2									○				
328.1																		
2.2	Silty CLAY, trace to some sand, some gravel Very Stiff to Hard Brown Moist (TILL)		4	SS	18		328							○				
			5	SS	22		327							○	—	—		16 13 30 41
							326											
	Grey		6	SS	23									○				
							325											
			7	SS	74		324							○				
							323											
	Stiff		8	SS	15		322							○				
321.4																		
9.0	SILT Dense Grey Wet		9	SS	50		321							○				
320.3																		

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

ONTMT4S2 MTO-11375(GINTDATA).GPJ 2017TEMPLATE(MTO).GDT 6/1/20

# RECORD OF BOREHOLE No SH16-03

3 OF 3

METRIC

GWP# 408-88-00 LOCATION Shantz Station Road, MTM NAD 83 Zone 10: N 4 818 710.2 E 233 448.4 ORIGINATED BY GA  
 DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA  
 DATUM Geodetic DATE 2017.07.27 - 2017.07.27 LATITUDE 43.504907 LONGITUDE -80.382385 CHECKED BY JPL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
	Continued From Previous Page													
	Silty SAND, some gravel, some clay Very Dense Brown Wet (TILL)	0 4 0 4			.175		310							10 47 30 13
309.0	Boulders	0 4												
21.4	END OF BOREHOLE AT 21.4m UPON REFUSAL ON BOULDERS OR PROBABLE BEDROCK. WATER LEVEL AT 12.2m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE AND AUGER CUTTINGS TO SURFACE.		17	SS	100/ .050		309							

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No SH16-04

1 OF 3

METRIC

GWP# 408-88-00 LOCATION Shantz Station Road, MTM NAD 83 Zone 10: N 4 818 684.8 E 233 480.5 ORIGINATED BY GA  
 DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA  
 DATUM Geodetic DATE 2017.07.24 - 2017.07.24 LATITUDE 43.504681 LONGITUDE -80.381984 CHECKED BY JPL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE									WATER CONTENT (%)
330.0	GROUND SURFACE							20	40	60	80	100		20	40	60	
0.0	Gravelly SAND, some silt Compact Brown Moist (FILL)		1	SS	23								○				
329.3																	
0.7	Sandy SILT, some gravel Compact Brown Moist		2	SS	21		329						○				
328.6																	
1.4	Silty CLAY, trace sand, trace to some gravel Stiff to Very Stiff Brown Moist (TILL)		3	SS	10		328						○				
			4	SS	27								○				
	sandy Hard to Very Stiff		5	SS	37		327						○				
							326										
	Grey		6	SS	26		325						○				
324.1																	
5.9	Silty SAND, some gravel Very Dense Grey Moist		7	SS	100/.075		324						○				
323.2																	
6.9	Hard						323										
			8	SS	80		322						○				
	Wet		9	SS	66		321						○				

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

ONTMT4S2 MTO-11375(GINTDATA)GPJ 2017TEMPLATE(MTO).GDT 6/1/20

# RECORD OF BOREHOLE No SH16-04

2 OF 3

METRIC

GWP# 408-88-00 LOCATION Shantz Station Road, MTM NAD 83 Zone 10: N 4 818 684.8 E 233 480.5 ORIGINATED BY GA  
DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA  
DATUM Geodetic DATE 2017.07.24 - 2017.07.24 LATITUDE 43.504681 LONGITUDE -80.381984 CHECKED BY JPL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
							20	40	60	80	100	20	40	60		
	Continued From Previous Page															
	Silty CLAY, some sand, trace to some gravel Hard to Very Stiff Brown to Grey Wet (TILL)		10	SS	38											7 17 46 30
			11	SS	17											
			12	SS	21											
	Moist		13	SS	18											
313.7																
16.3	Silty SAND, some gravel to gravelly, trace to some clay Very Dense Grey Wet (TILL)		14	SS	83											15 46 26 13
			15	SS	100/ 225											
			16	SS	100/											

Continued Next Page

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

ONTMT4S2 MTO-11375(GINTDATA).GPJ 2017TEMPLATE(MTO).GDT 6/1/20

RECORD OF BOREHOLE No SH16-04

3 OF 3

METRIC

GWP# 408-88-00 LOCATION Shantz Station Road, MTM NAD 83 Zone 10: N 4 818 684.8 E 233 480.5 ORIGINATED BY GA  
DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA  
DATUM Geodetic DATE 2017.07.24 - 2017.07.24 LATITUDE 43.504681 LONGITUDE -80.381984 CHECKED BY JPL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
	Continued From Previous Page																
308.5	Silty SAND, some gravel to gravelly, trace to some clay Very Dense Grey Wet (TILL)		17	SS	100/											23 47 23 7	
21.5	END OF BOREHOLE AT 21.5m. WATER LEVEL AT 12.2m 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) 2017.12.18 - - Piezometer was damaged				.175												



# RECORD OF BOREHOLE No SH16-05

1 OF 1

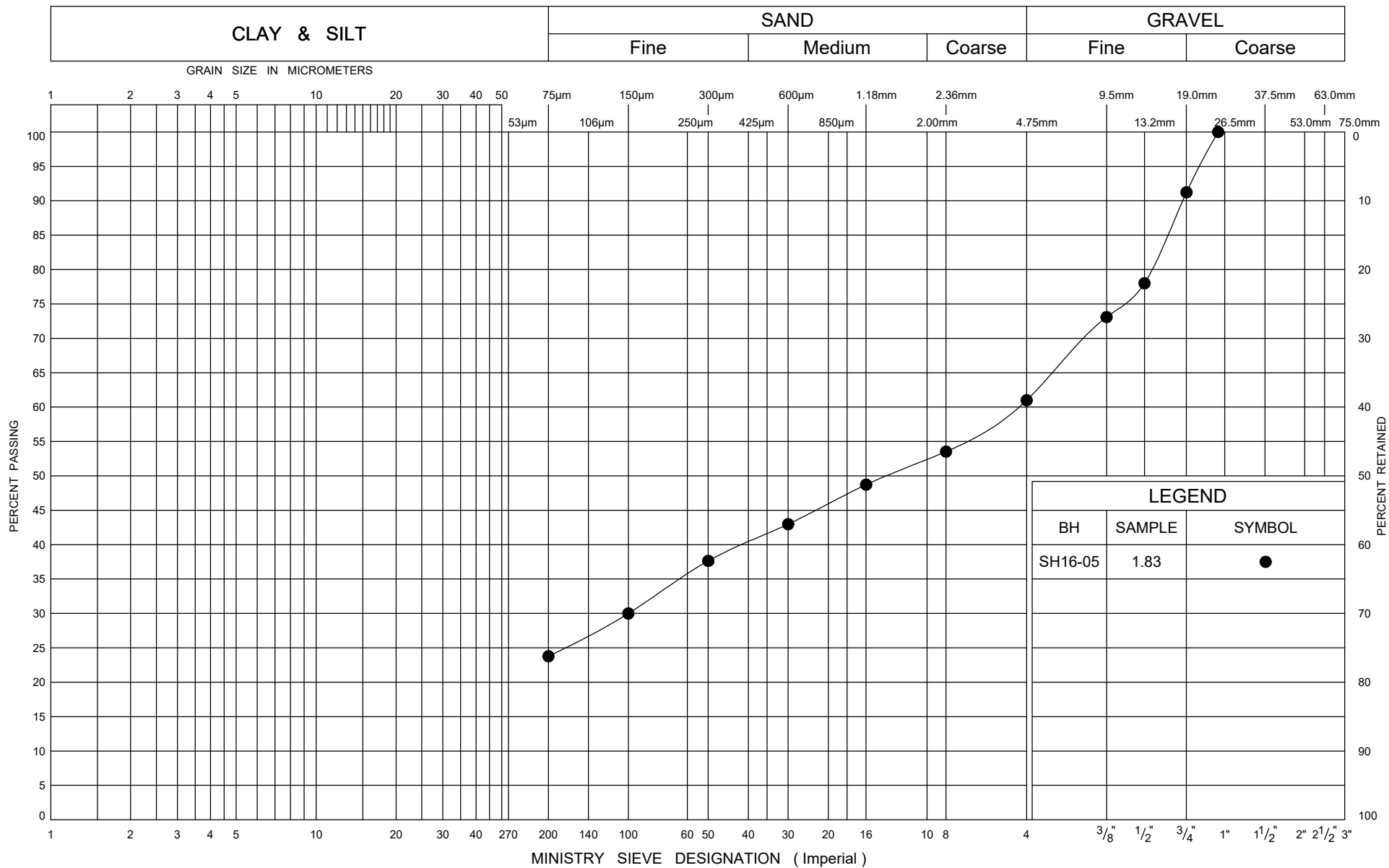
METRIC

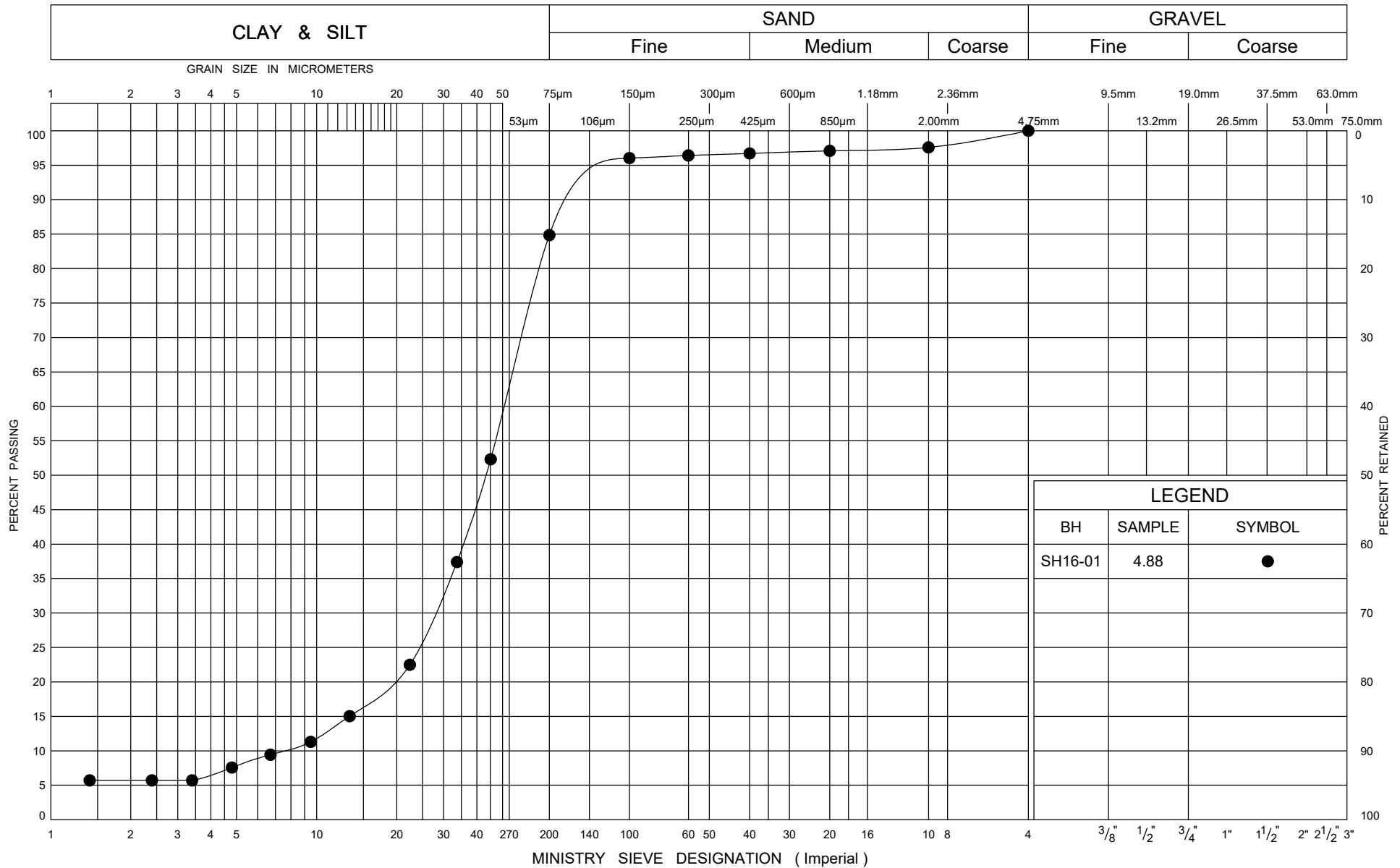
GWP# 408-88-00 LOCATION Shantz Station Road, MTM NAD 83 Zone 10: N 4 818 663.2 E 233 472.9 ORIGINATED BY GA  
 DIST HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA  
 DATUM Geodetic DATE 2017.06.30 - 2017.06.30 LATITUDE 43.504486 LONGITUDE -80.382076 CHECKED BY JPL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
329.5	GROUND SURFACE													
0.0	Gravelly SAND Dense Brown Moist (FILL)		1	SS	36		329							
0.7	Silty CLAY, some sand, some gravel Stiff Brown Moist (TILL)		2	SS	12									
328.1							328							
1.4	SAND and GRAVEL, some silt and clay Compact Brown Moist		3	SS	14									
327.3							327							
2.2	trace to some gravel, trace sand Very Stiff		4	SS	22									
							326							
							325							
	Hard		6	SS	75		324							
323.6														
5.9	Silty SAND, some gravel to gravelly, trace clay Very Dense Grey Wet (TILL)		7	SS	50									
323.4														
6.1	END OF BOREHOLE AT 6.1m UPON AUGER REFUSAL. BOREHOLE DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE AND AUGER CUTTINGS TO SURFACE.				.050									

ONTMT4S2 MTO-11375(GINTDATA)\GPJ 2017TEMPLATE(MTO).GDT 6/1/20

ONTARIO MOT GRAIN SIZE 2 MTO-11375(GINTDATA)\GPJ\_ONTARIO MOT.GDT 6/1/20





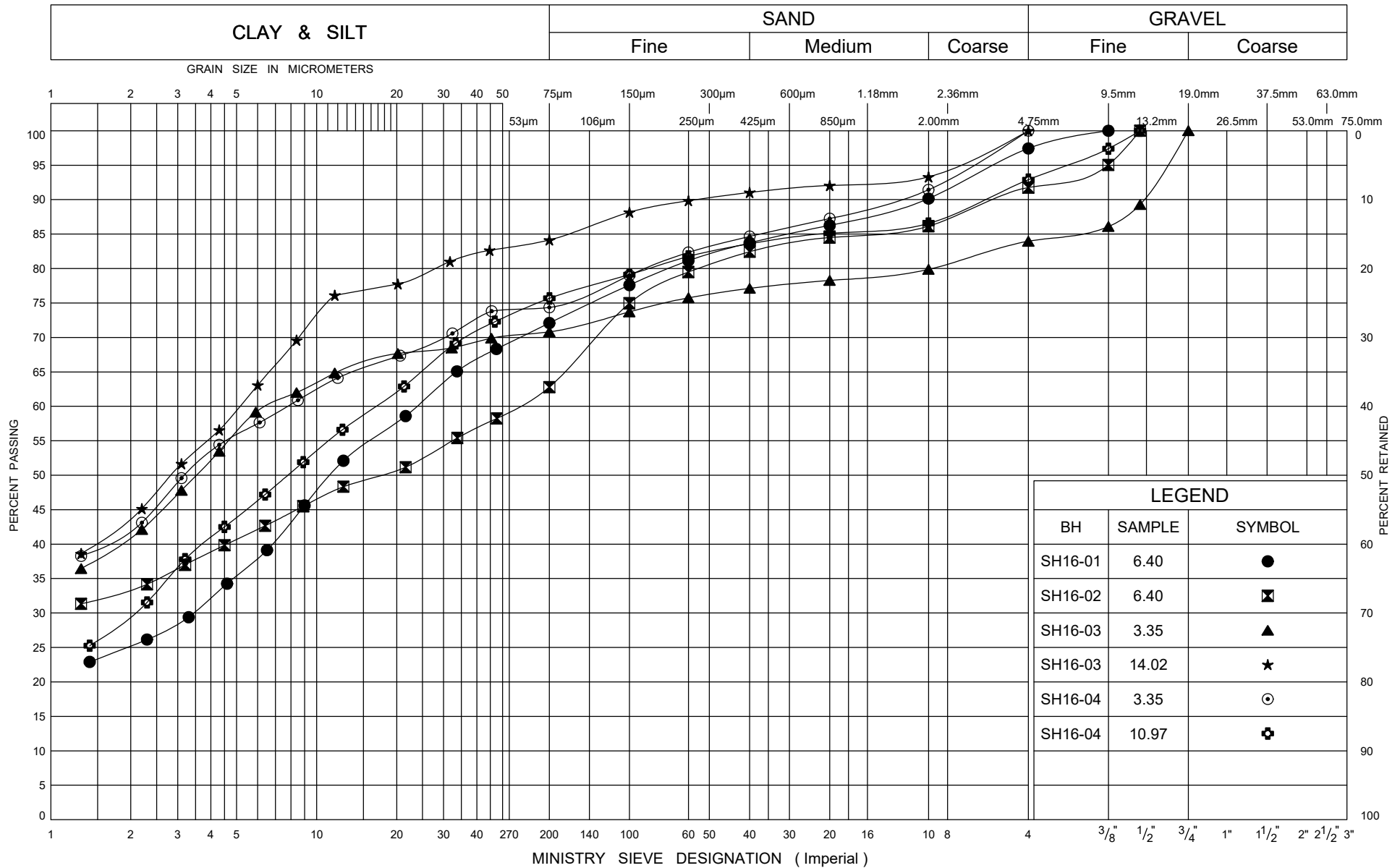
Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION SILT

FIG No A2

W P 408-88-00

Shantz Station Road



Ministry of  
Transportation

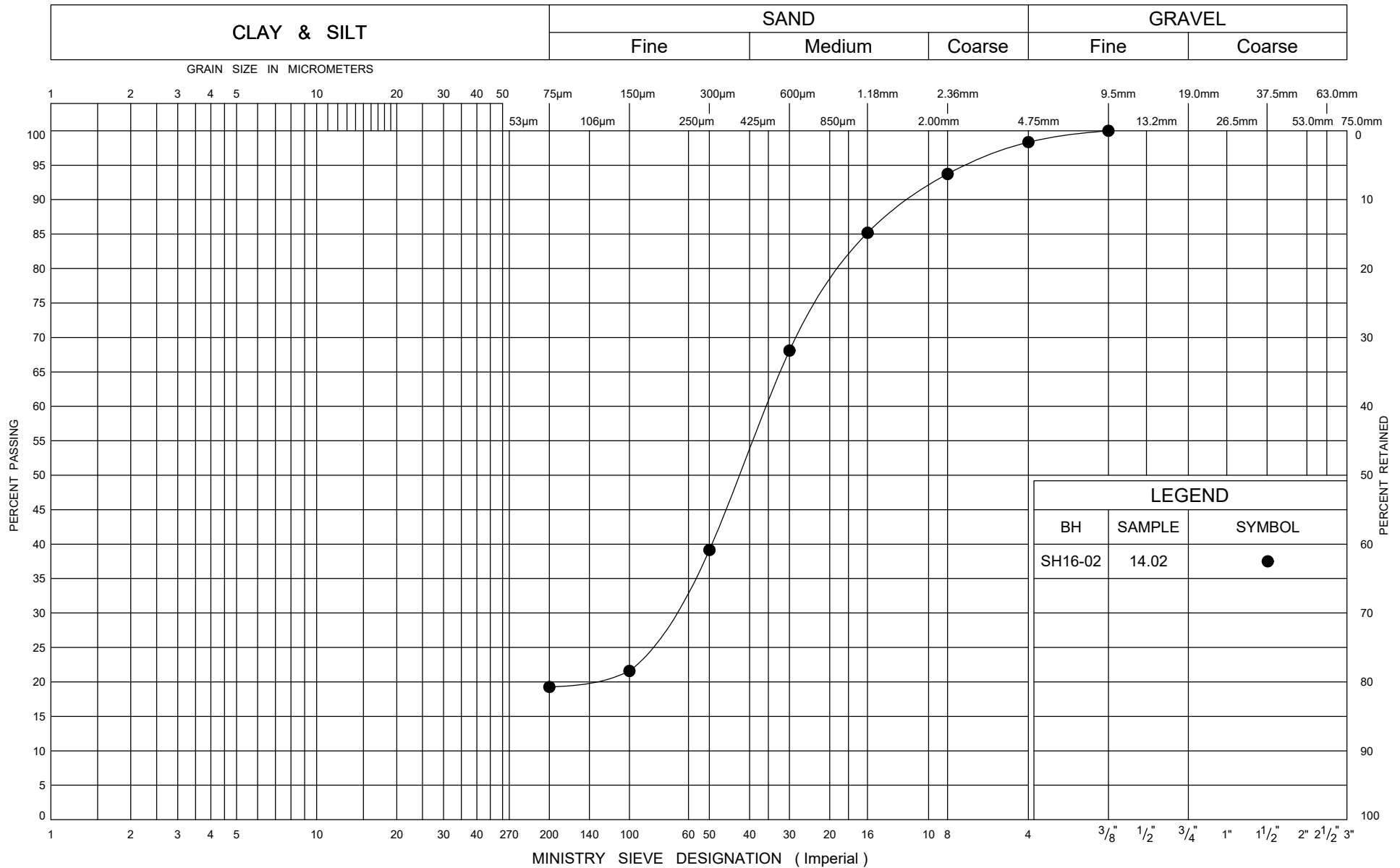
## GRAIN SIZE DISTRIBUTION

Silty CLAY TILL

FIG No C3

W P 408-88-00

Shantz Station Road



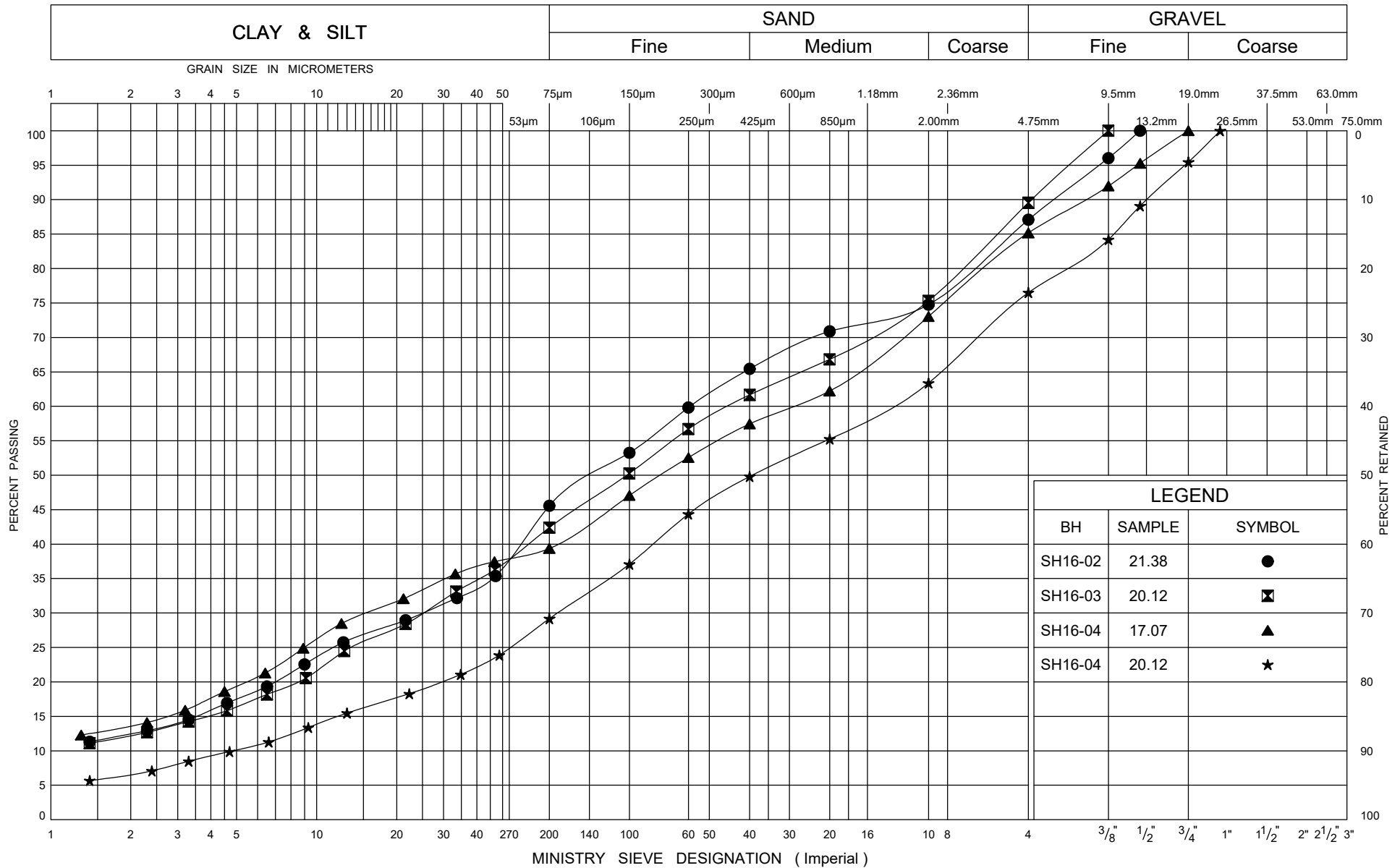
Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION SAND

FIG No A4

W P 408-88-00

Shantz Station Road



Ministry of  
Transportation

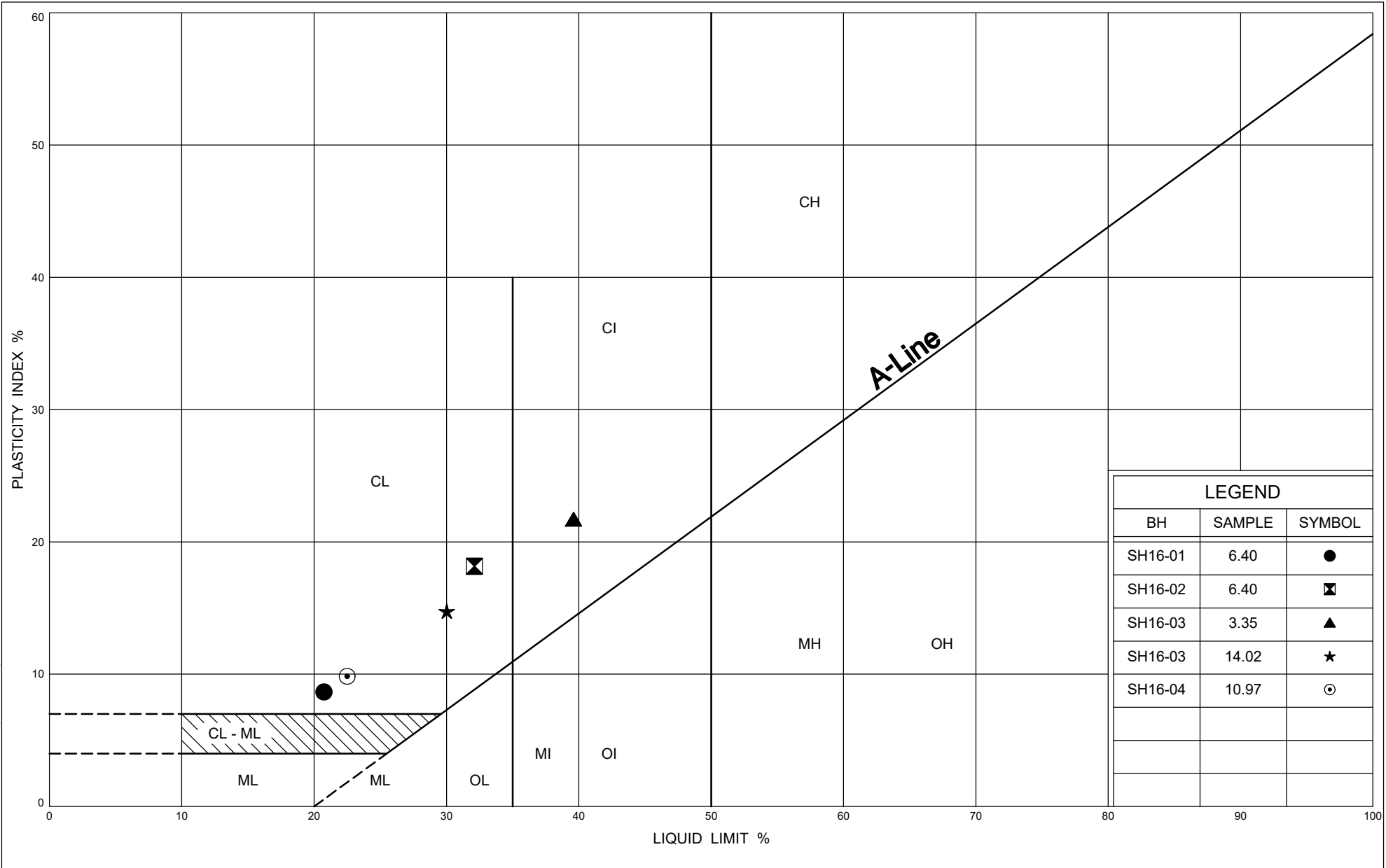
## GRAIN SIZE DISTRIBUTION

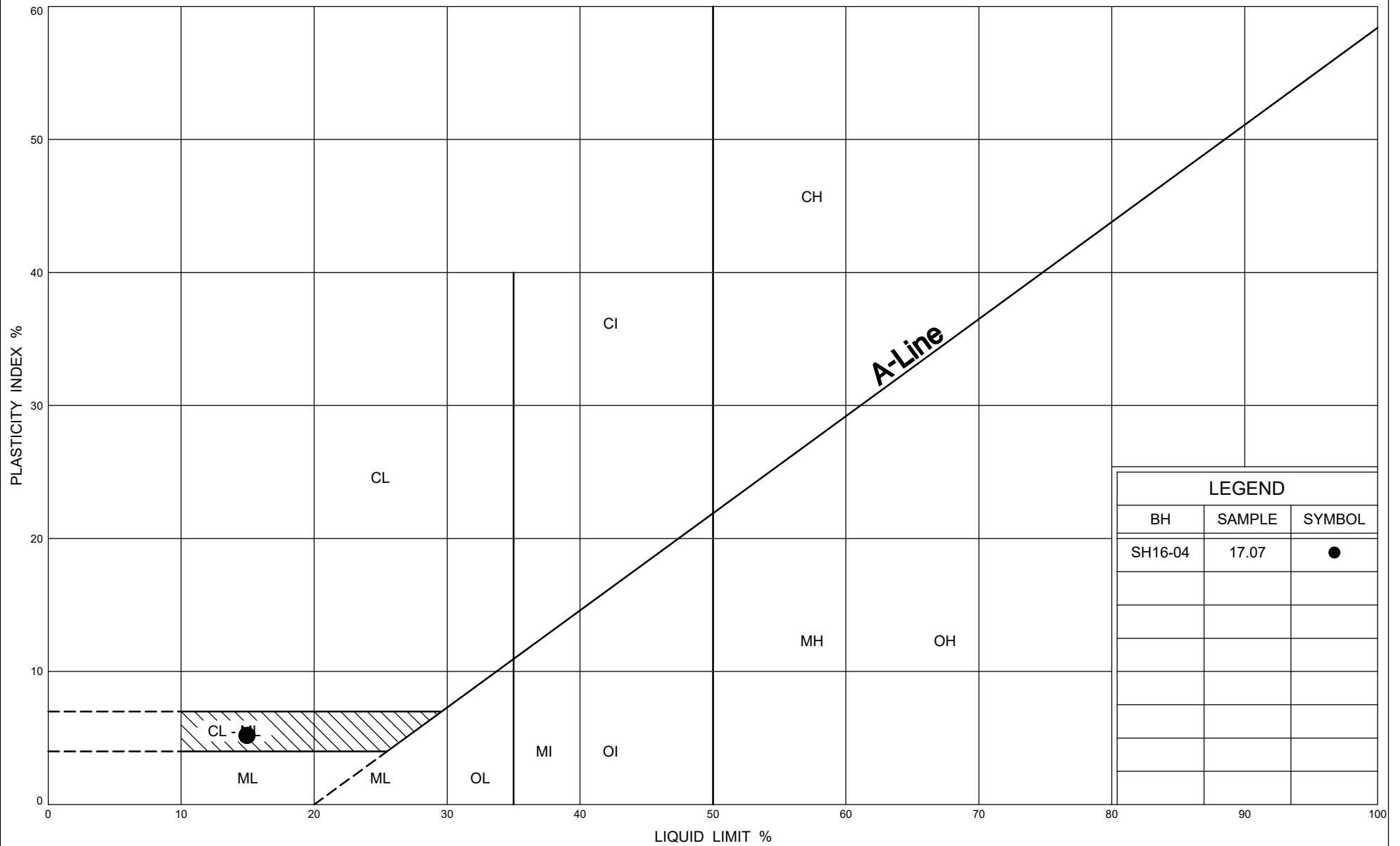
### Silty SAND TILL

FIG No A5

W P 408-88-00

Shantz Station Road





Ministry of  
Transportation

## PLASTICITY CHART

Silty SAND TILL

FIG No G7

W P 408-88-00

Shantz Station Road





**THURBER ENGINEERING LTD.**

**POINT LOAD TEST SHEET**  
**ASTM D5731-08**

**Job No:** 11375  
**Client:**  
**Project Name:** Hwy 7 Kitchener  
**Core Size:** NQ **BH No :** SH-16-02

**Date Drilled:** July 26/17  
**Date Tested:** Sep 5/17  
**Tester:** JZ  
**Reviewed by:** WM

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	21.9	D	13.4	47.0	220.4	5.6	134.0	Limestone	Very Strong
2	1	22.4	A	10.0	47.0	52.8	3.2	75.9	Limestone	Strong
3	1	22.9	D	7.1	47.0	147.6	2.9	70.7	Limestone	Strong
4	2	23.5	D	11.5	47.0	110.3	4.8	114.8	Limestone	Very Strong
5	2	24.2	A	19.4	47.0	60.6	5.5	132.5	Limestone	Very Strong
6	2	24.8	D	16.8	47.0	180.5	7.0	167.9	Limestone	Very Strong
7										
8										
9										
10										
11										
12										
13										
14										
15										
16										
17										
18										
19										
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31										
32										
33										
34										

- \* It is ideal to perform axial test on core specimens with D/L ratio of  $1.1 \pm 0.1$   
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
- \* Diametral Test should have  $0.7 \times D$  on either side of test point.
- \* Correlation factor to obtain UCS values is 24.



## FINAL REPORT

CA14400-MAR18 R

11375

Prepared for

**Thurber Engineering Ltd.**

## First Page

### CLIENT DETAILS

Client Thurber Engineering Ltd.

Address 103, 2010 Winston Park Drive  
Oakville, ON  
L6H 5R7.

Contact Rocio Reyna

Telephone 905-829-8666 x 263

Facsimile

Email rreyna@thurber.ca

Project 11375

Order Number

Samples Soil (12)

### LABORATORY DETAILS

Project Specialist Deanna Edwards, B.Sc, C.Chem

Laboratory SGS Canada Inc.

Address 185 Concession St., Lakefield ON, K0L 2H0

Telephone 705-652-2000

Facsimile 705-652-6365

Email deanna.edwards@sgs.com

SGS Reference CA14400-MAR18

Received 03/19/2018

Approved 03/23/2018

Report Number CA14400-MAR18 R

Date Reported 03/23/2018

### COMMENTS

Temperature of Sample upon Receipt: 2 degrees C

Cooling Agent Present: Yes

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





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# FINAL REPORT

CA14400-MAR18 R

Client: Thurber Engineering Ltd.

Project: 11375

Project Manager: Rocío Reyna

Samplers: Kamil Feszak

PACKAGE: - Corrosivity Index (SOIL)

Sample Number	5	6	7	8	9	10	11	12
Sample Name	BS16-04 SS4	GH16-04 SS8	RC16-02 SS3	CR04 SS3	EB 16-03 SS5	SP16-04 SS7	CV16-01 SS3	GRB16-10 SS4
Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
Sample Date	21/03/2018							

Parameter	Units	RL		Result	Result	Result	Result	Result	Result	Result
Corrosivity Index										
Corrosivity Index	none	1		4.0	3.0	4.0	4.0	3.0	5.5	4.0
Soil Redox Potential	mV	-		343	324	305	294	332	271	228
Sulphide	%	0.02		< 0.02	< 0.02	< 0.02	< 0.02	< 0.02	0.06	< 0.02
pH	no unit	0.05		9.08	8.73	8.47	8.63	8.60	8.49	8.78
Resistivity (calculated)	ohms.cm	-9999		3860	3390	4630	3950	6100	2800	7520

PACKAGE: - Corrosivity Index (SOIL)

Sample Number	13	14	15	16
Sample Name	HC16-05 SS3	TR04-SS5	SH16-04 SS4	GRB16-21 SS4
Sample Matrix	Soil	Soil	Soil	Soil

Parameter	Units	RL		Result	Result	Result	Result
Corrosivity Index							
Corrosivity Index	none	1		4.0	4.0	3.0	4.0
Soil Redox Potential	mV	-		314	250	265	246
Sulphide	%	0.02		< 0.02	< 0.02	< 0.02	< 0.02
pH	no unit	0.05		9.06	8.98	9.11	8.91
Resistivity (calculated)	ohms.cm	-9999		7810	10100	6940	8200



# FINAL REPORT

CA14400-MAR18 R

Client: Thurber Engineering Ltd.

Project: 11375

Project Manager: Rocío Reyna

Samplers: Kamil Feszak

## PACKAGE: - General Chemistry (SOIL)

Sample Number	5	6	7	8	9	10	11	12
Sample Name	BS16-04 SS4	GH16-04 SS8	RC16-02 SS3	CR04 SS3	EB 16-03 SS5	SP16-04 SS7	CV16-01 SS3	GRB16-10 SS4
Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
Sample Date	21/03/2018							

Parameter	Units	RL		Result	Result	Result	Result	Result	Result	Result
-----------	-------	----	--	--------	--------	--------	--------	--------	--------	--------

### General Chemistry

Conductivity	uS/cm	2		259	295	216	253	164	357	133	118
--------------	-------	---	--	-----	-----	-----	-----	-----	-----	-----	-----

## PACKAGE: - General Chemistry (SOIL)

Sample Number	13	14	15	16
Sample Name	HC16-05 SS3	TR04-SS5	SH16-04 SS4	GRB16-21 SS4
Sample Matrix	Soil	Soil	Soil	Soil

Parameter	Units	RL		Result	Result	Result	Result
-----------	-------	----	--	--------	--------	--------	--------

### General Chemistry

Conductivity	uS/cm	2		128	99	144	122
--------------	-------	---	--	-----	----	-----	-----

## PACKAGE: - Metals and Inorganics (SOIL)

Sample Number	5	6	7	8	9	10	11	12
Sample Name	BS16-04 SS4	GH16-04 SS8	RC16-02 SS3	CR04 SS3	EB 16-03 SS5	SP16-04 SS7	CV16-01 SS3	GRB16-10 SS4
Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
Sample Date	21/03/2018							

Parameter	Units	RL		Result	Result	Result	Result	Result	Result	Result
-----------	-------	----	--	--------	--------	--------	--------	--------	--------	--------

### Metals and Inorganics

Sulphate	µg/g	0.4		140	92	11	69	6.5	356	68	22
----------	------	-----	--	-----	----	----	----	-----	-----	----	----

## PACKAGE: - Metals and Inorganics (SOIL)

Sample Number	13	14	15	16
Sample Name	HC16-05 SS3	TR04-SS5	SH16-04 SS4	GRB16-21 SS4
Sample Matrix	Soil	Soil	Soil	Soil

Parameter	Units	RL		Result	Result	Result	Result
-----------	-------	----	--	--------	--------	--------	--------

### Metals and Inorganics

Sulphate	µg/g	0.4		22	2.4	15	11
----------	------	-----	--	----	-----	----	----



# FINAL REPORT

CA14400-MAR18 R

Client: Thurber Engineering Ltd.

Project: 11375

Project Manager: Rocío Reyna

Samplers: Kamil Feszak

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

Sample Number	5	6	7	8	9	10	11	12
Sample Name	BS16-04 SS4	GH16-04 SS8	RC16-02 SS3	CR04 SS3	EB 16-03 SS5	SP16-04 SS7	CV16-01 SS3	GRB16-10 SS4
Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
Sample Date	21/03/2018							

Parameter	Units	RL		Result	Result	Result	Result	Result	Result	Result	Result
Other (ORP)											
Chloride	µg/g	0.4		34	50	12	71	4.8	7.6	13	67

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

Sample Number	13	14	15	16
Sample Name	HC16-05 SS3	TR04-SS5	SH16-04 SS4	GRB16-21 SS4
Sample Matrix	Soil	Soil	Soil	Soil

Parameter	Units	RL		Result	Result	Result	Result
Other (ORP)							
Chloride	µg/g	0.4		71	22	94	68

## PACKAGE: - PHCs (SOIL)

Sample Number	5	6	7	8	9	10	11	12
Sample Name	BS16-04 SS4	GH16-04 SS8	RC16-02 SS3	CR04 SS3	EB 16-03 SS5	SP16-04 SS7	CV16-01 SS3	GRB16-10 SS4
Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
Sample Date	21/03/2018							

Parameter	Units	RL		Result	Result	Result	Result	Result	Result	Result	Result
PHCs											
Moisture Content	%	0.1		14.5	0.2	12.8	8.6	1.2	19.9	5.5	8.7

## PACKAGE: - PHCs (SOIL)

Sample Number	13	14	15	16
Sample Name	HC16-05 SS3	TR04-SS5	SH16-04 SS4	GRB16-21 SS4
Sample Matrix	Soil	Soil	Soil	Soil

Parameter	Units	RL		Result	Result	Result	Result
PHCs							
Moisture Content	%	0.1		12.4	7.1	2.7	10.8



# FINAL REPORT

CA14400-MAR18 R

## QC SUMMARY

### Anions by IC

Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-IENVIIC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Chloride	DIO0288-MAR18	µg/g	0.4	<0.4	2	20	100	80	120	101	75	125
Sulphate	DIO0288-MAR18	µg/g	0.4	<0.4	15	20	98	80	120	96	75	125

### Carbon/Sulphur

Method: ASTM E1915-07A | Internal ref.: ME-CA-IENVIARD-LAK-AN-020

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Sulphide	ECS0025-MAR18	%	0.02	<0.02	ND	20	111	80	120			

### Conductivity

Method: SM 2510 | Internal ref.: ME-CA-IENVIEWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0284-MAR18	uS/cm	2	< 2	1	10	99	90	110	NA		





FINAL REPORT

CA14400-MAR18 R

QC SUMMARY

pH  
Method: SM 4500 | Internal ref.: ME-CA-|ENVIEWL-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0284-MAR18	no unit	0.05	NA	1		101			NA		

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

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

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

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

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

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

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

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

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

## LEGEND

## FOOTNOTES

**NSS** Insufficient sample for analysis.

**RL** Reporting Limit.

↑ Reporting limit raised.

↓ Reporting limit lowered.

**NA** The sample was not analysed for this analyte

**ND** Non Detect

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

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

SGS provides criteria information (such as regulatory or guideline limits and summary of limit exceedances) as a service. Every attempt is made to ensure the criteria information in this report is accurate and current, however, it is not guaranteed. Comparison to the most current criteria is the responsibility of the client and SGS assumes no responsibility for the accuracy of the criteria levels indicated. This document is issued, on the Client's behalf, by the Company under its General Conditions of Service available on request and accessible at [http://www.sgs.com/terms\\_and\\_conditions.htm](http://www.sgs.com/terms_and_conditions.htm). The Client's attention is drawn to the limitation of liability, indemnification and jurisdiction issues defined therein. Any other holder of this document is advised that information contained hereon reflects the Company's findings at the time of its intervention only and within the limits of Client's instructions, if any. The Company's sole responsibility is to its Client and this document does not exonerate parties to a transaction from exercising all their rights and obligations under the transaction documents.

This report must not be reproduced, except in full. This report supersedes all previous versions.

-- End of Analytical Report --



# Request for Laboratory Services and CHAIN OF CUSTODY

No:

Page 1 of 2

SGS Environmental Services - 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

Received By:

*Enak Agsey*  
Received Date (mm/dd/yyyy): 03/15/2018 (mm/dd/yyyy)  
Received Time: 11:00 AM

Received By (signature):

Custody Seal Present:

Custody Seal Intact:

Laboratory Information Section - Lab use only

Cooling Agent Present:

Temperature Upon Receipt (°C)

LAB LIMS #:

## REPORT INFORMATION

Company: *Thurber Eng.*

Contact: *Rocio Palomares Reyna*

Address: *103-2010 Winstonpark Dr.*

*Oakville, ON*

Phone: *905-829-8666 x260*

Fax:

Email: *rreynae@thurber.ca*

## INVOICE INFORMATION

☒ (same as Report Information)

Company:

Contact:

Address:

Phone:

Email:

## PROJECT INFORMATION

Quotation #:

Project #:

P.O. #:

Site Location/ID:

## TURNAROUND TIME (TAT) REQUIRED

☒ Regular TAT (5-7 days) 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:

## REGULATIONS

### Regulation 153 (2011):

☐ Table 1 ☐ Res/Park ☐ Soil Texture:

☐ Table 2 ☐ Ind/Com ☐ Coarse

☐ Table 3 ☐ Agri/Other ☐ Medium

☐ Table ☐ 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

### SAMPLE IDENTIFICATION

DATE SAMPLED

TIME SAMPLED

# OF BOTTLES

MATRIX

1 TR-04 -SSS

2 SH16-0A S54

3 GRB16-2A S54

4

5

6

7

8

9

10

## ANALYSIS REQUESTED

COMMENTS:

Field Filtered (F)

Preserved (P)

Observations/Comments/Special Instructions

Sampled By (NAME): *KAMIL FESZAK*

Relinquished by (NAME): *Sarah Hashidai*

Signature:

Signature:

Date:

Date:

Pink Copy - Client

Yellow & White Copy - SGS





# Request for Laboratory Services and CHAIN OF CUSTODY

SGS Environmental Services

- 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 2 of 2

Received By:

Enoch Forster

Received Date (mm/dd/yyyy): 03/17/2018

Received Time: 11:00 AM

## Laboratory Information Section - Lab use only

Received By (signature):

Custody Seal Present: ☒

Custody Seal Intact: ☒

Cooling Agent Present: ☒

Temperature Upon Receipt (°C): 7.0, 7.1, 7.2

CA 14400-MAR-18

LAB LIMS #:

5x3

## REPORT INFORMATION

Company: Thumber Eng.

Contact: Rogo Palomares Reyna

Address: 103-2010 Winston Dr. Oakville, ON

Phone: 905-824-8666

Fax: 905-824-8666

Email: preyna@thumber.ca

## INVOICE INFORMATION

☒ (same as Report Information)

Company:

Contact:

Address:

Phone:

Fax:

Email:

## PROJECT INFORMATION

Quotation #:

Project #:

P.O. #:

Site Location/ID:

## TURNAROUND TIME (TAT) REQUIRED

TAT's are quoted in business days (exclude statutory holidays & weekends).

Samples received after 3pm or on weekends : TAT begins the next business day

☐ Regular TAT (5-7days) ☐ 1 Day ☐ 2 Days ☐ 3-4 Days

RUSH TAT (Additional Charges May Apply)

PLEASE CONFIRM RUSH FEASIBILITY WITH SGS REPRESENTATIVE PRIOR TO SUBMISSION

Specify Due Date: Rush Confirmation ID:

## REGULATIONS

Regulation 153 (2011):

☐ Table 1 ☐ Res/Park ☐ Soil Texture:

☐ Table 2 ☐ Ind/Com ☐ Coarse

☐ Table 3 ☐ Agri/Other ☐ Medium

☐ Table ☐ 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

### SAMPLE IDENTIFICATION

1 B516-04 S54

2 B516-04 S58

3 RC16-02 S53

4 LR04 S53

5 EB16-03 S55

6 ~~EB16-03 S55~~

7 SP16-04 S57

8 CV16-01 S53

9 GRB16-10 S54

10 HC16-05 S53

DATE SAMPLED

March 21/2017

Oct 1/2017

Mar 1/2017

Dec 1/2016

Oct 1/2016

May 1/2017

June 1/2017

Dec 1/2017

Aug 1/2017

TIME SAMPLED

# OF BOTTLES

MATRIX

Soil

Soil

Soil

Soil

Soil

Soil

Soil

Soil

Soil

Soil

## ANALYSIS REQUESTED

COMMENTS:  
Field Filtered (F)  
Preserved (P)

Observations/Comments/Special Instructions

Sampled By (NAME): KAMIL FESZAK

Relinquished by (NAME): Sarah Hashemi

Signature:

Signature:

Date: 03/19/18 (mm/dd/yy)

Date: (mm/dd/yy)

Pink Copy - Client

Yellow & White Copy - SGS

Revision #: 1.0

Date of Issue: 01 June, 2014



# SAMPLE INTEGRITY REPORT

Project Number: 11375

ONTARIO REGULATION 153/04

SGS Sample ID CA 14400 - MAR 18

Date / Time Sampled See CoFC

Client Sample ID See CoFC

ALL

## Sample Submission General Sample Integrity Violations

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

## Sample Specific Sample Integrity Violations

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

## Sediment Log

- |  |                          |                          |                          |                          |                          |                          |                          |
|--|--------------------------|--------------------------|--------------------------|--------------------------|--------------------------|--------------------------|--------------------------|
| Groundwater samples contain visible sediment / particulate                       | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |
| Groundwater contains greater than 1cm of sediment / particulate matter in bottle | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |

## Additional Comments/Remarks:

No issues upon receipt



Initials:

KH



## **Appendix B**

### **Record of Borehole Sheets and Laboratory Test Results (Previous Investigation)**

# RECORD OF BOREHOLE No 08-157

1 OF 3

METRIC

G.W.P. 408-88-00 LOCATION N 4 818 744.22 E 233 432.47 ORIGINATED BY SLL  
 HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM  
 DATUM Geodetic DATE 2008.05.30 - 2008.06.02 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								20 40 60 80 100					
								20 40 60 80 100					
329.6													
0.0	TOPSOIL: (75mm)												
0.1	Silty SAND, trace rootlets Loose Dark Brown Moist (FILL)		1	SS	6								
328.2													
1.4	Silty, sandy CLAY, trace gravel Firm Brown (FILL)		2	SS	4								
327.4													
2.2	Silty CLAY, some sand, trace gravel Very Stiff to Hard Brown to Grey (TILL)		3	SS	20								
			4	SS	30								
			5	SS	28								
			6	SS	100								
			7	SS	54								
321.1													
8.5	Silty CLAY Very Stiff Grey		8	SS	26								

Continued Next Page

+ <sup>3</sup> X <sup>3</sup> Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 08-157

2 OF 3

METRIC

G.W.P. 408-88-00 LOCATION N 4 818 744 22 E 233 432 47 ORIGINATED BY SLL  
 HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM  
 DATUM Geodetic DATE 2008.05.30 - 2008.06.02 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	
	Continued From Previous Page							SHEAR STRENGTH kPa					
								○ UNCONFINED + FIELD VANE					
								● QUICK TRIAXIAL x LAB VANE					
								WATER CONTENT (%)					
								20 40 60 80 100	20 40 60				
319.3 10.3	Silty CLAY Very Stiff Grey						319						0 16 80 4
	SILT, some fine sand, trace clay Compact Grey Wet		9	SS	27		318						
317.7 11.9	Clayey SILT, trace to some sand Hard Grey (TILL)						317						
			10	SS	73		316						0 9 70 21
			11	SS	87		315						
314.8 14.8	Silty CLAY, trace sand, trace gravel, occasional cobbles Very Stiff Grey (TILL)						314						0 5 30 65
			12	SS	22		313						
			13	SS	22		312						
			14	SS	29		311						
310.4 19.2	Sandy SILT, some clay, trace gravel Very Dense Grey (TILL)						310						

Continued Next Page

+<sup>3</sup> x<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No 08-157

3 OF 3

METRIC

G.W.P. 408-88-00 LOCATION N 4 818 744.22 E 233 432.47 ORIGINATED BY SLL  
 HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM  
 DATUM Geodetic DATE 2008.05.30 - 2008.06.02 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
	Continued From Previous Page																
308.8	Sandy SILT, some clay, trace gravel Very Dense Grey (TILL)		15	SS	65											GR 5 SA 40 SI 41 CL 14	
20.8	END OF BOREHOLE AT 20.8m. AUGER REFUSAL AT 20.8m ON PROBABLE BEDROCK OR BOULDER. BOREHOLE DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE BENSEAL TO 1.5m, HOLEPLUG FROM 1.5m TO 0.6m, THEN CUTTINGS TO SURFACE.																

# RECORD OF BOREHOLE No 08-158

1 OF 3

METRIC

G.W.P. 408-88-00 LOCATION N 4 818 713.04 E 233 462.61 ORIGINATED BY SLL  
 HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM  
 DATUM Geodetic DATE 2008.05.22 - 2008.05.23 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT  Y  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								20 40 60 80 100				
								20 40 60 80 100				
330.0												
0.0	SAND, some gravel, mixed with clay, topsoil stained Dark Brown (FILL)						330					
329.5												
0.5	Silty CLAY, some sand to sandy, trace gravel, occasional cobbles Stiff to Hard Brown (TILL)		1	SS	14		329					
			2	SS	40		328					
			3	SS	67		327					
			4	SS	74		326					
	becoming Grey											
			5	SS	46		325					
			6	SS	83		324					
			7	SS	61		323					
	clay pockets											
							322					
321.2												
8.8	SILT, some sand, trace gravel, trace clay Compact Grey Wet						321					
320.7												
9.3	Clayey SILT, trace sand, trace gravel Very Stiff Grey: (TILL)		8	SS	20							

Continued Next Page

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 08-158

2 OF 3

METRIC

G.W.P. 408-86-00 LOCATION N 4 818 713.04 E 233 462.61 ORIGINATED BY SLL  
 HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM  
 DATUM Geodetic DATE 2008.05.22 - 2008.05.23 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	×						
								● QUICK TRIAXIAL	×	LAB VANE						
	Continued From Previous Page						20 40 60 80 100									
	Clayey SILT, trace sand Hard Grey (TILL)		9	SS	51									0 5 75 20		
			10	SS	125									0 7 78 15		
316.7																
13.3	Silty CLAY, trace sand, trace gravel Hard to Very Stiff Grey (TILL)		11	SS	55											
			12	SS	31											
	Slow augering		13	SS	33									1 6 35 58		
312.3																
17.7	Sandy SILT, some clay, trace gravel Very Dense Grey (TILL)		14	SS	100/ .175									4 42 39 15		
			15	SS	100/											

Continued Next Page

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 08-158

3 OF 3

METRIC

G.W.P. 408-88-00 LOCATION N 4 818 713.04 E 233 462.61 ORIGINATED BY SLL  
 HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM  
 DATUM Geodetic DATE 2008.05.22 - 2008.05.23 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE									
	Continued From Previous Page																
	Sandy SILT, some clay, trace gravel Very Dense Grey (TILL)					.125											
308.6			16	SS	100/	.100											
21.4	END OF BOREHOLE AT 21.4m, AUGER REFUSAL ON PROBABLE BEDROCK OR BOULDER. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.  WATER LEVEL READINGS: DATE DEPTH(m) ELEV.(m) 2008.05.28 6.8 323.2 2008.06.02 8.8 321.2 2008.07.15 9.0 321.0 2008.08.14 8.4 321.6																

# RECORD OF BOREHOLE No 08-159

1 OF 3

METRIC

G.W.P. 408-88-00 LOCATION N 4 818 685.97 E 233 461.96 ORIGINATED BY SLL  
 HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM  
 DATUM Geodetic DATE 2008.06.03 - 2008.06.03 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
329.2												
0.0	TOPSOIL: (200 mm)											
0.2	Silty CLAY, trace gravel, occasional topsoil staining Stiff Brown (FILL)		1	SS	11		329					
327.8							328					
1.4	Silty CLAY, some sand, trace gravel Hard Brown (TILL)		2	SS	63							
			3	SS	42		327					
			4	SS	64		326					
325.1												
4.1	Sandy SILT, some clay, trace to some gravel Very Dense Grey (TILL)		5	SS	100/ 150		325					
							324					
323.6												
5.6	Silty CLAY, trace sand Very Stiff to Hard Grey (TILL)		6	SS	47		323					
							322					
			7	SS	29		321					
	occasional sand seams						320					
			8	SS	46							

Continued Next Page

+<sup>3</sup> × 3<sup>3</sup> Numbers refer to 20  
Sensitivity 15 10 5 10 (%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No 08-159

3 OF 3

METRIC

G.W.P. 408-88-00 LOCATION N 4 818 685.97 E 233 461.96 ORIGINATED BY SLL  
 HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM  
 DATUM Geodetic DATE 2008.06.03 - 2008.06.03 CHECKED BY RPR

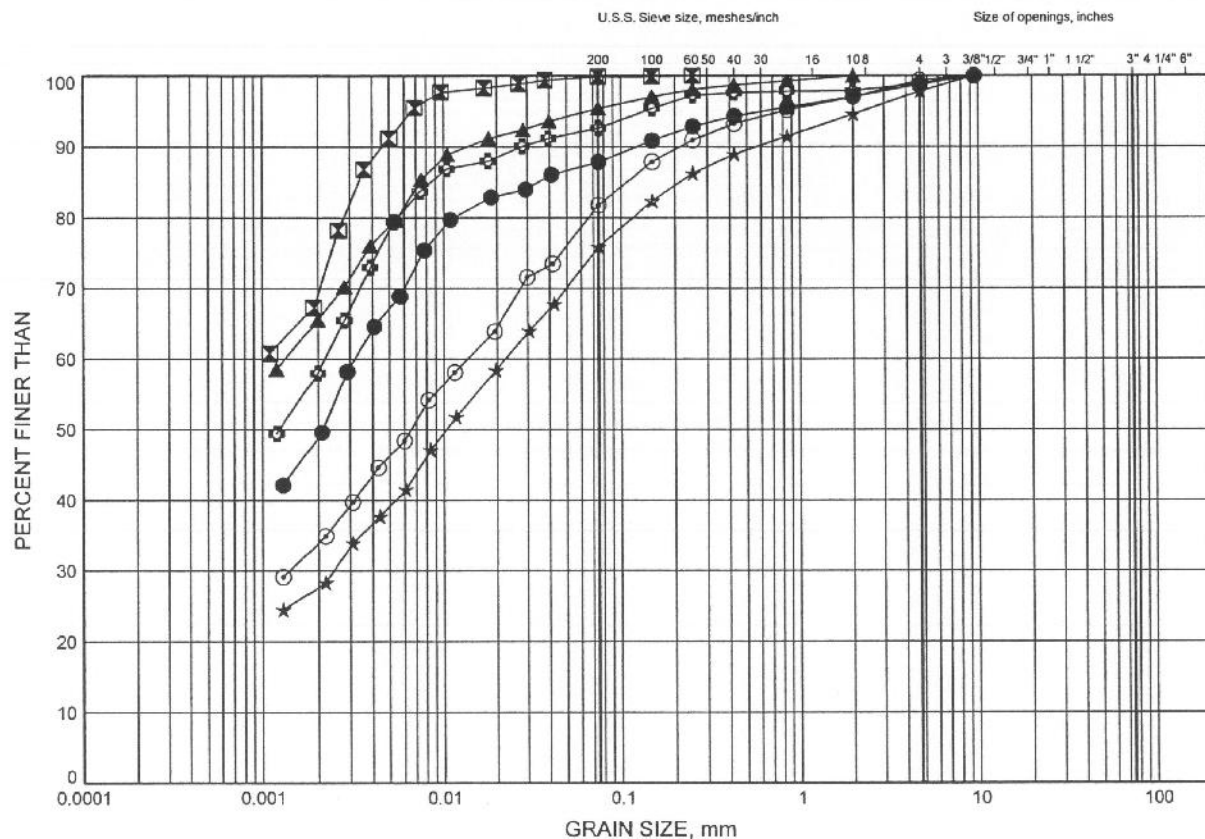
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W <sub>p</sub> W W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
20.0	<p>Continued From Previous Page</p> <p>END OF BOREHOLE AT 20.0m.                      AUGER REFUSAL ON PROBABLE                      BEDROCK OR BOULDER.                      BOREHOLE DRY UPON                      COMPLETION OF DRILLING.                      BOREHOLE BACKFILLED WITH                      GROUT TO 15.0m, HOLEPLUG TO                      3.0m, THEN AUGER CUTTINGS TO                      SURFACE.</p>		15	SS	100/ .025						



# Highway 7 - New GRAIN SIZE DISTRIBUTION

FIGURE B1

## Silty Clay Till



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-157	3.35	326.25
⊠	08-157	9.45	320.15
▲	08-157	15.54	314.06
☆	08-158	3.35	326.65
⊙	08-158	7.92	322.08
⊕	08-158	15.54	314.46



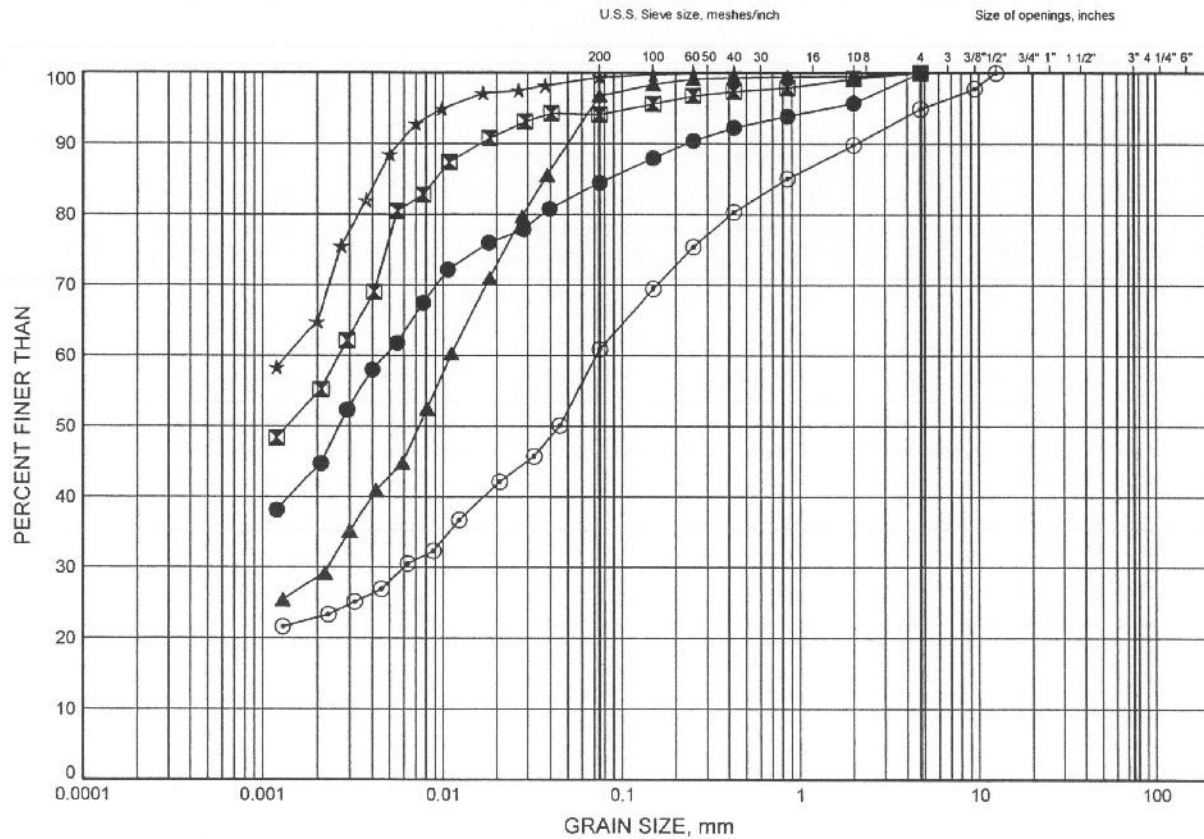
W.P.# 408-88-00  
Prepared By MFA  
Checked By RPR



# Highway 7 - New GRAIN SIZE DISTRIBUTION

FIGURE B2

## Silty Clay Till



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-159	2.59	326.61
⊠	08-159	6.40	322.80
▲	08-159	9.37	319.83
☆	08-159	14.02	315.18
⊙	08-159	15.49	313.71

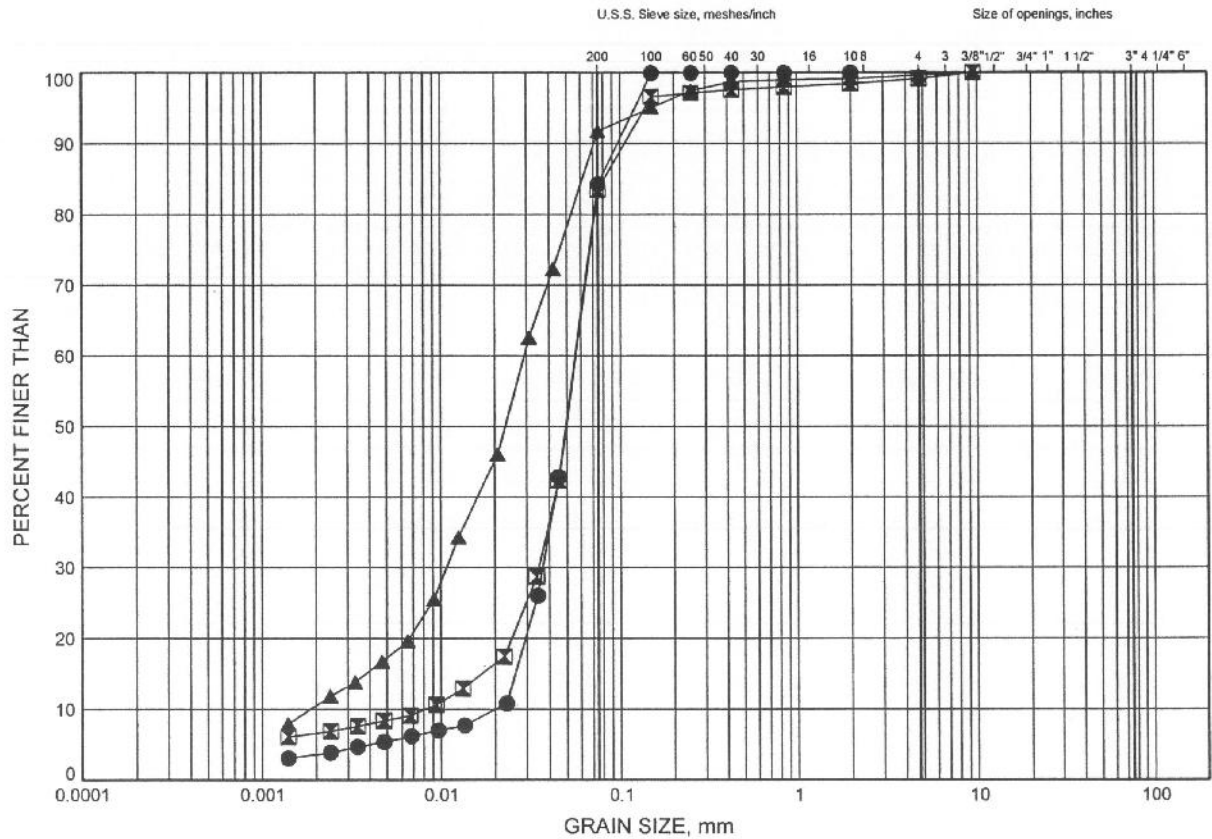


W.P.# 408-88-00  
Prepared By MFA  
Checked By RPR

# Highway 7 - New GRAIN SIZE DISTRIBUTION

FIGURE B3

## Silt / Silt Till



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

## LEGEND

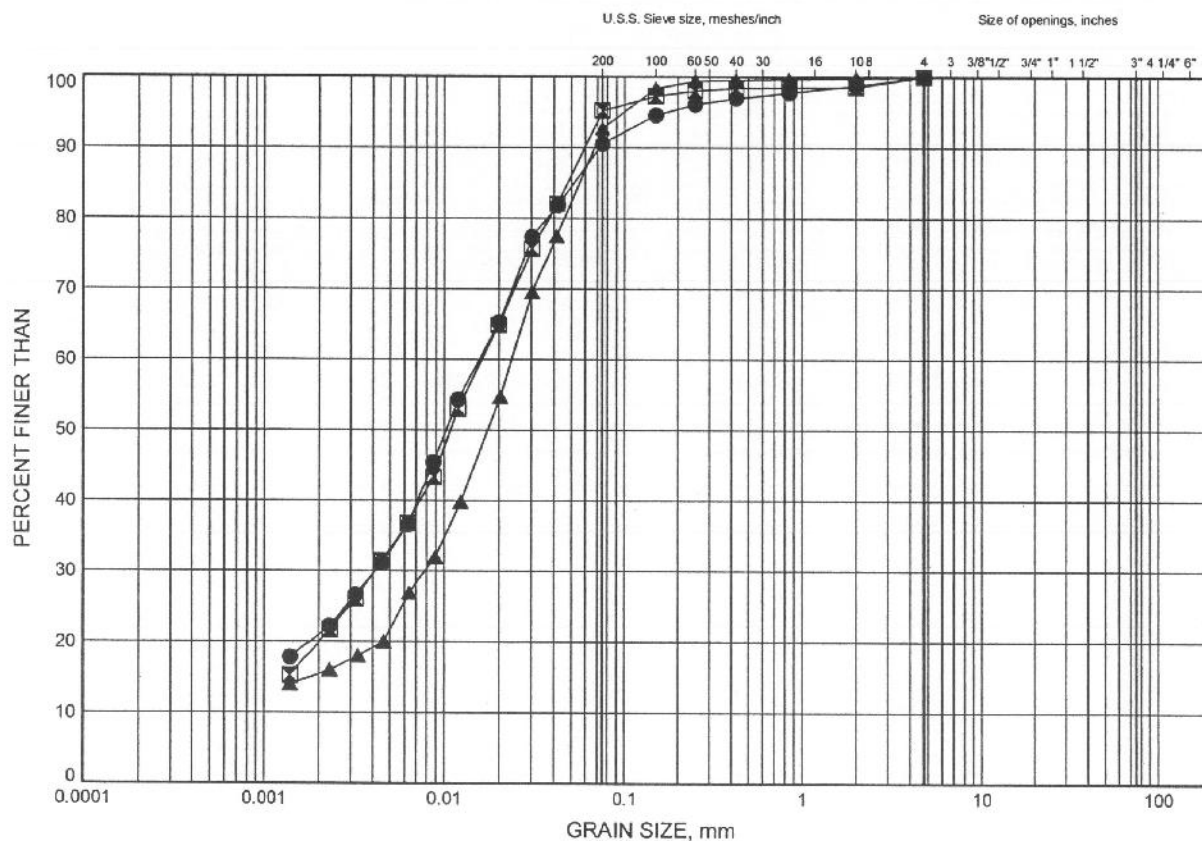
SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-157	10.97	318.63
■	08-158	9.24	320.76
▲	08-159	10.79	318.41



# Highway 7 - New GRAIN SIZE DISTRIBUTION

FIGURE B4

## Clayey Silt Till



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-157	14.02	315.58
■	08-158	10.97	319.03
▲	08-158	12.50	317.50

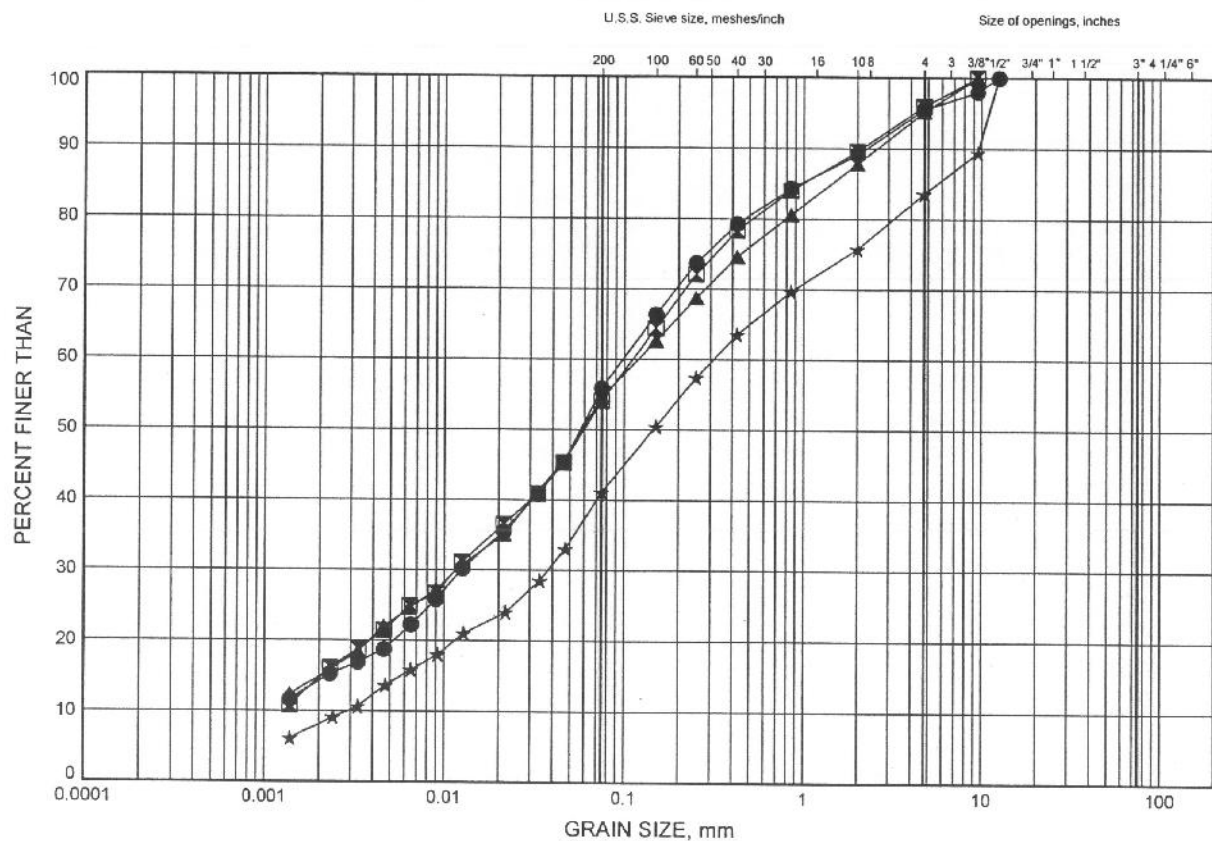


W.P.# 408-88-00  
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# Highway 7 - New GRAIN SIZE DISTRIBUTION

FIGURE B5

## Sandy Silt Till



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-157	20.04	309.56
⊠	08-158	18.30	311.70
▲	08-159	4.72	324.48
★	08-159	16.89	312.31

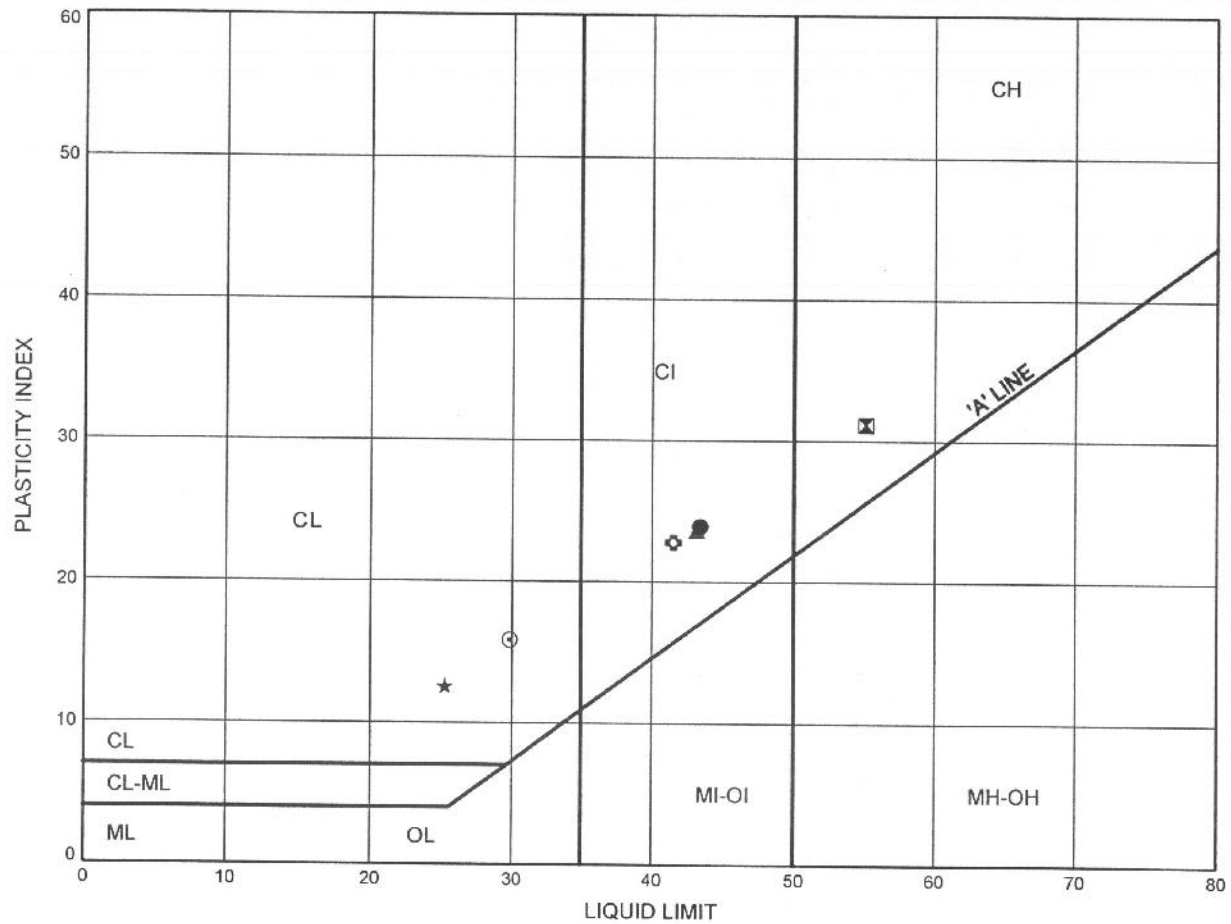


W.P.# 408-88-00  
Prepared By MFA  
Checked By RPR

# Highway 7 - New ATTERBERG LIMITS TEST RESULTS

FIGURE B6

Silty Clay Till



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-157	3.35	326.25
⊠	08-157	9.45	320.15
▲	08-157	15.54	314.06
★	08-158	3.35	326.65
⊙	08-158	7.92	322.08
⊗	08-158	15.54	314.46

Date August 2008  
 Project 408-88-00

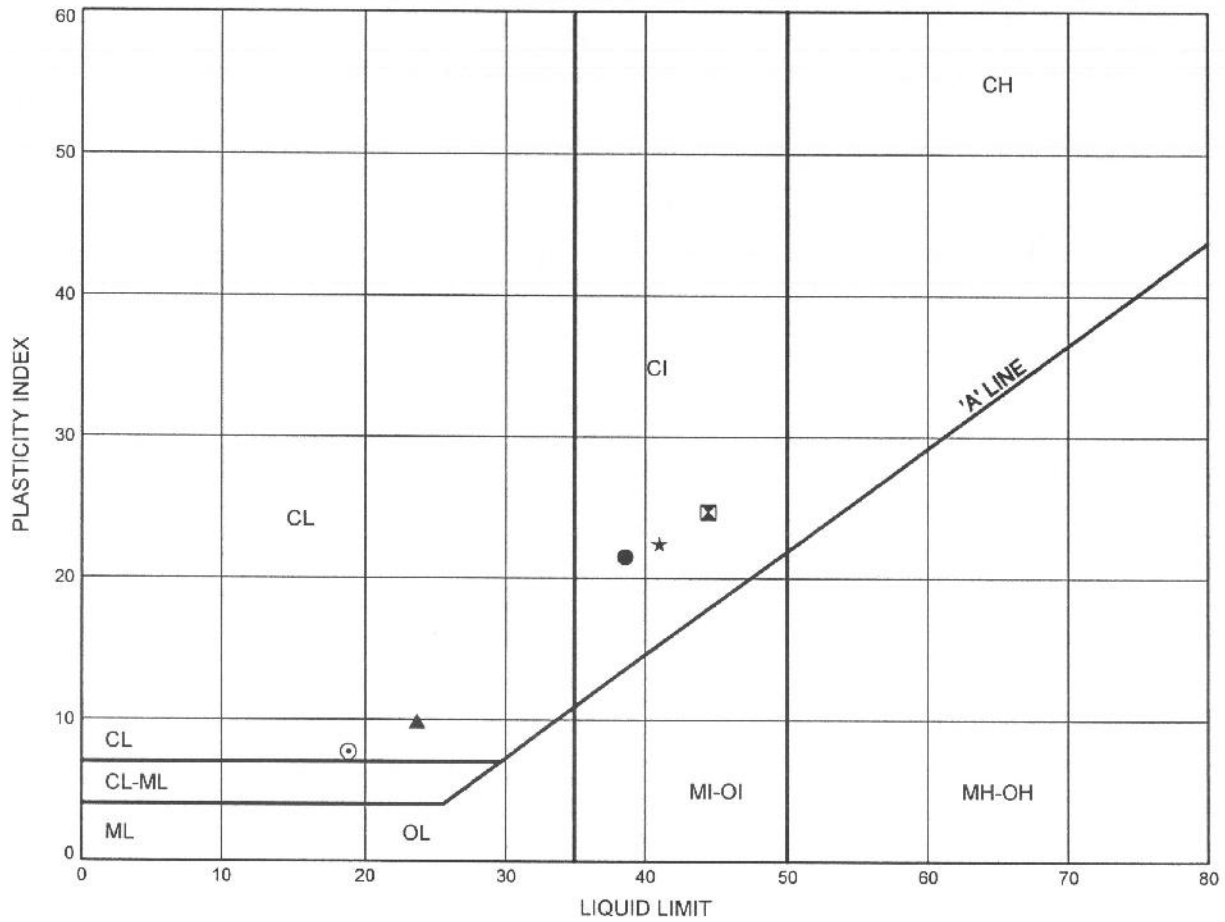


Prep'd MFA  
 Chkd. RPR

# Highway 7 - New ATTERBERG LIMITS TEST RESULTS

FIGURE B7

Silty Clay Till



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-159	2.59	326.61
⊠	08-159	6.40	322.80
▲	08-159	9.37	319.83
★	08-159	14.02	315.18
⊙	08-159	15.49	313.71



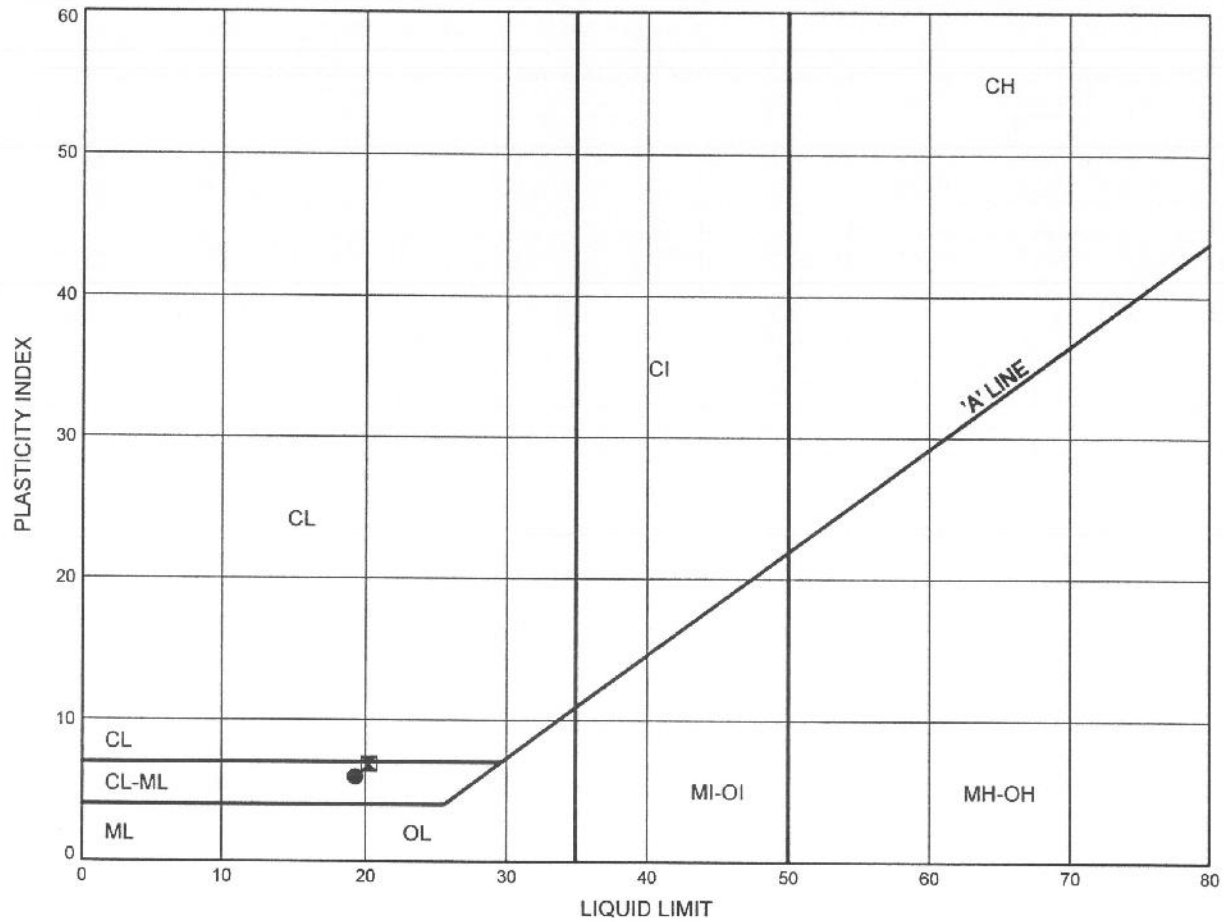
Date August 2008  
Project 408-88-00

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Chkd. RPR

# Highway 7 - New ATTERBERG LIMITS TEST RESULTS

FIGURE B8

Clayey Silt Till

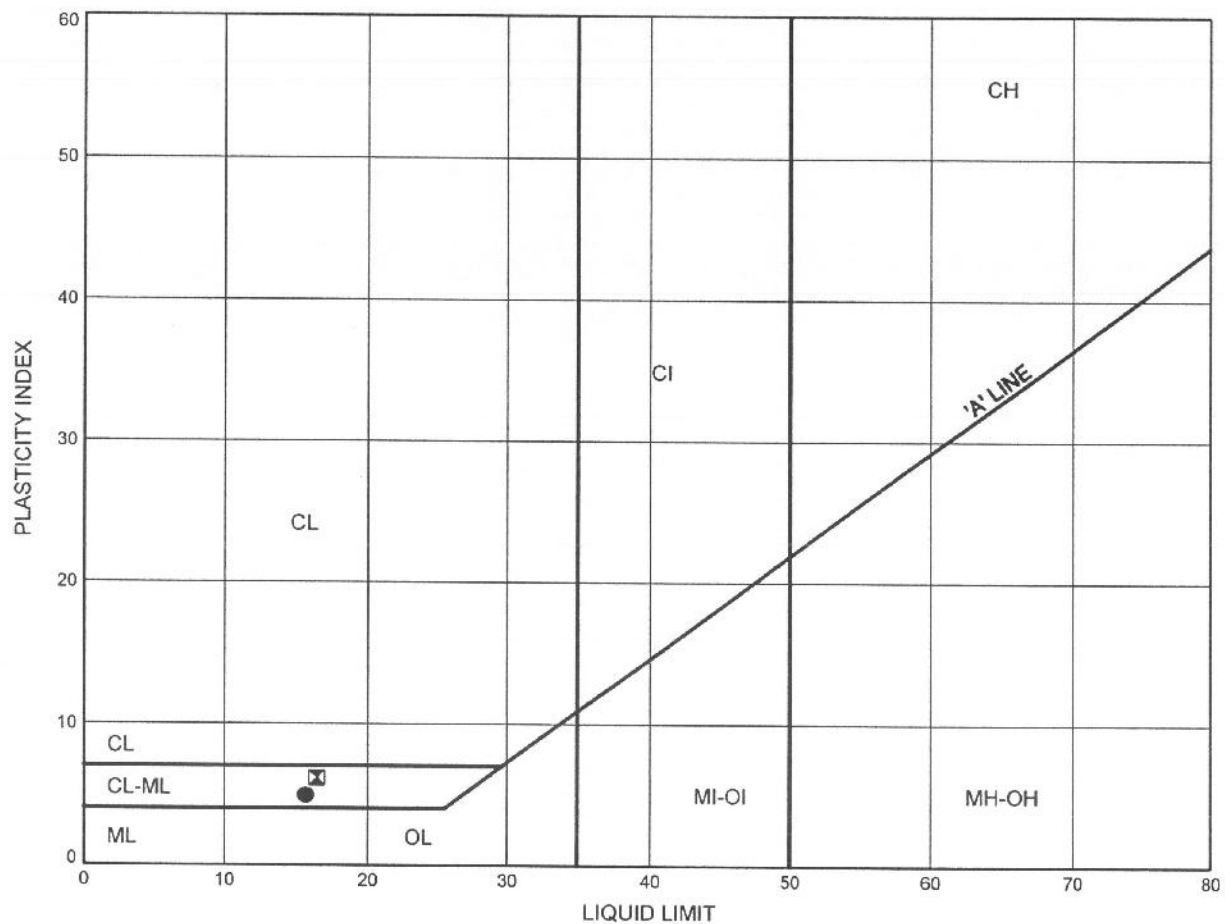


SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-157	14.02	315.58
⊠	08-158	10.97	319.03

# Highway 7 - New ATTERBERG LIMITS TEST RESULTS

FIGURE B9

Sandy Silt Till



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-157	20.04	309.56
⊠	08-158	18.44	311.56

THURBALT 6417R.GPJ 8/19/08

Date August 2008  
Project 408-88-00



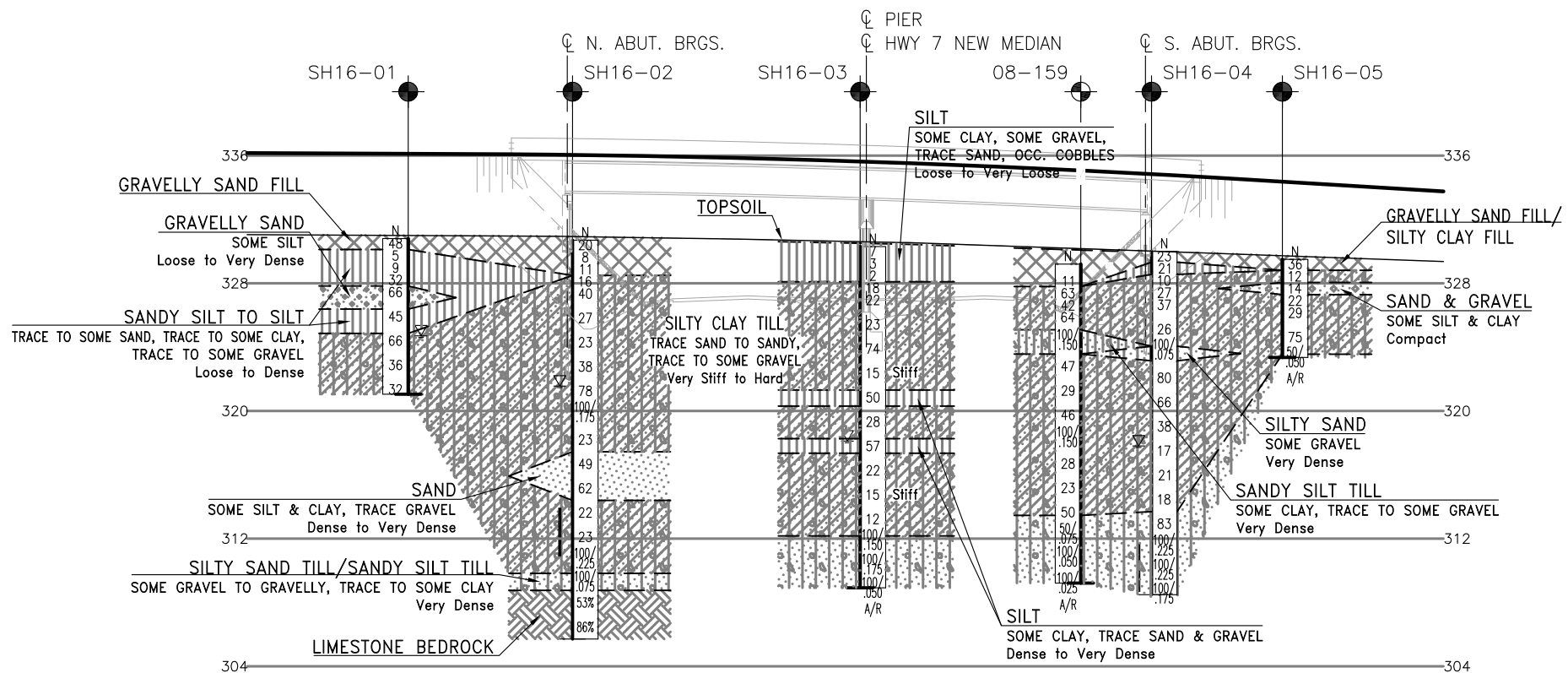
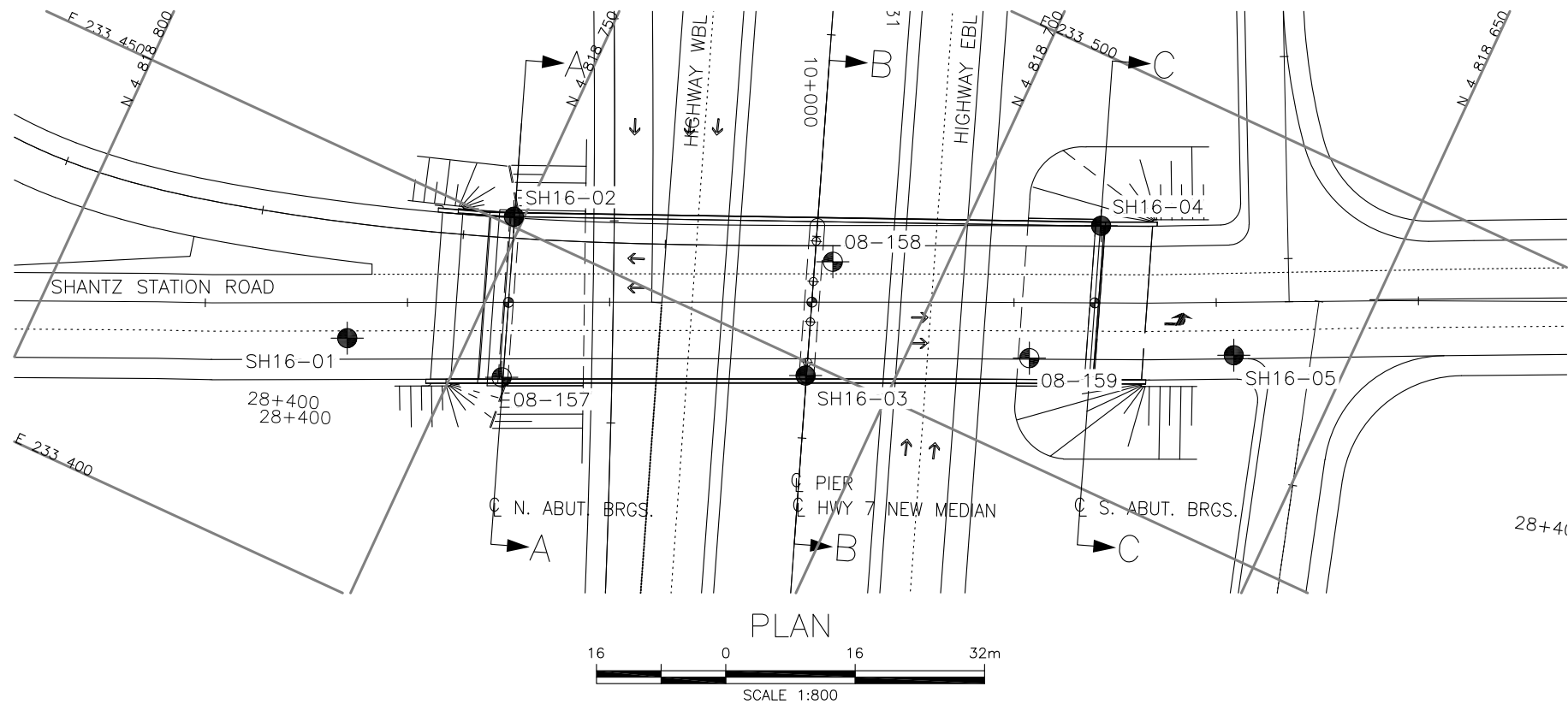
Prep'd MFA  
Chkd. RPR



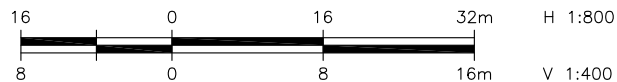


## **Appendix C**

### **Borehole Locations and Soil Strata Drawing**



PROFILE ALONG SHANTZ STATION ROAD



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

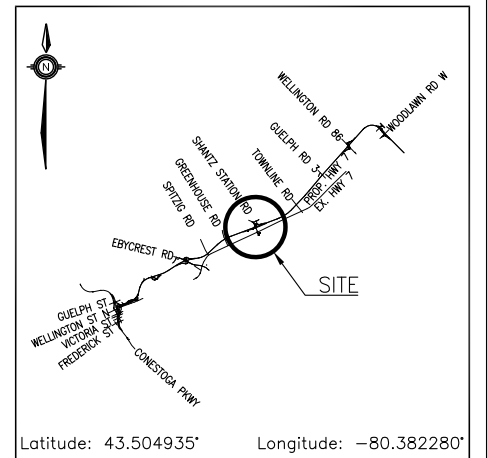


CONT No  
GWP No 408-88-00

HIGHWAY 7  
SHANTZ STATION ROAD  
PROPOSED UNDERPASS  
BOREHOLE LOCATIONS AND SOIL STRATA



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
SH16-01	330.8	4 818 763.6	233 428.9
SH16-02	330.7	4 818 751.2	233 451.1
SH16-03	330.3	4 818 710.2	233 448.4
SH16-04	330.0	4 818 684.8	233 480.5
SH16-05	329.5	4 818 663.2	233 472.9
08-157	329.6	4 818 744.2	233 432.5
08-158	330.0	4 818 713.0	233 462.6
08-159	329.2	4 818 686.0	233 462.0

NOTES

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

GEOCRES No. 40P9-61

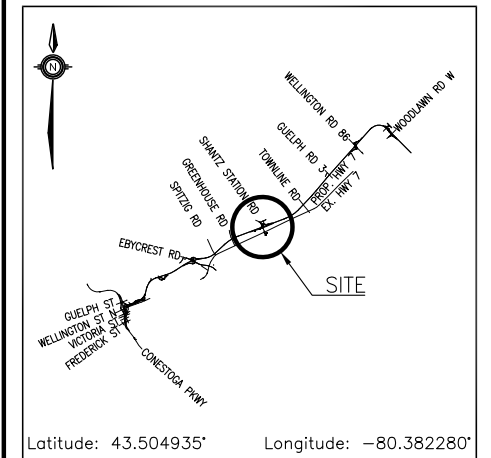
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METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT No  
GWP No 408-88-00

HIGHWAY 7  
SHANTZ STATION ROAD  
PROPOSED UNDERPASS  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



KEYPLAN

LEGEND

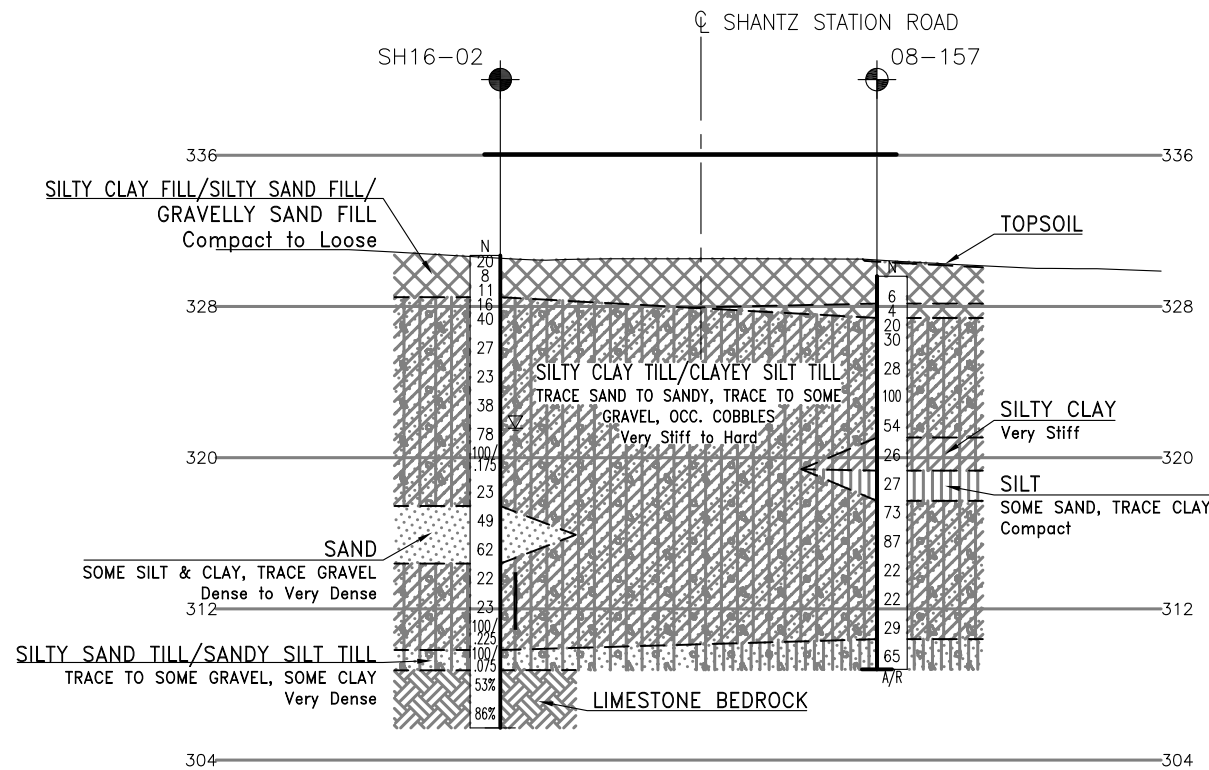
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- ⊙ Borehole (2008 Investigation)
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60° Cone, 475J/blow)
- PH Pressure, Hydraulic
- ☼ Water Level
- ☼ Head Artesian Water
- ☼ Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SH16-01	330.8	4 818 763.6	233 428.9
SH16-02	330.7	4 818 751.2	233 451.1
SH16-03	330.3	4 818 710.2	233 448.4
SH16-04	330.0	4 818 684.8	233 480.5
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08-158	330.0	4 818 713.0	233 462.6
08-159	329.2	4 818 686.0	233 462.0

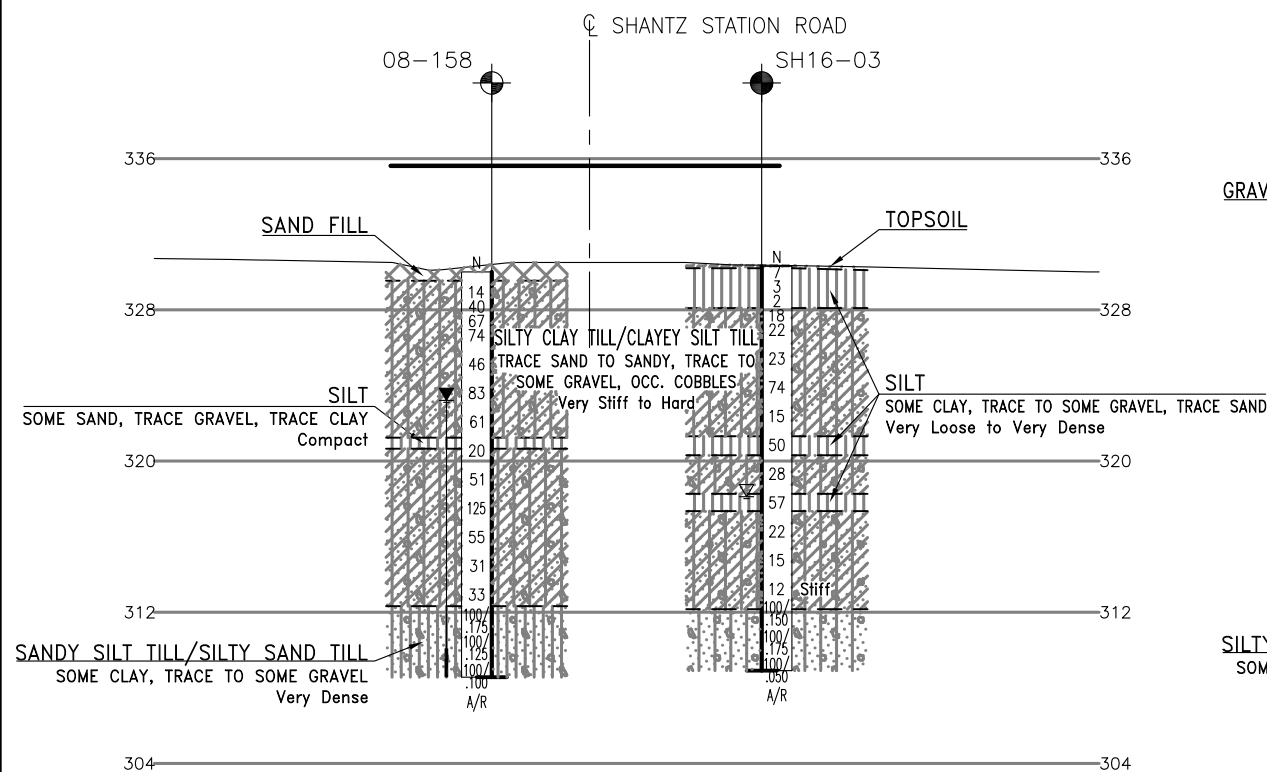
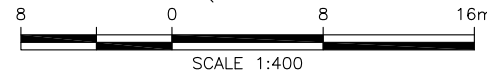
-NOTES-

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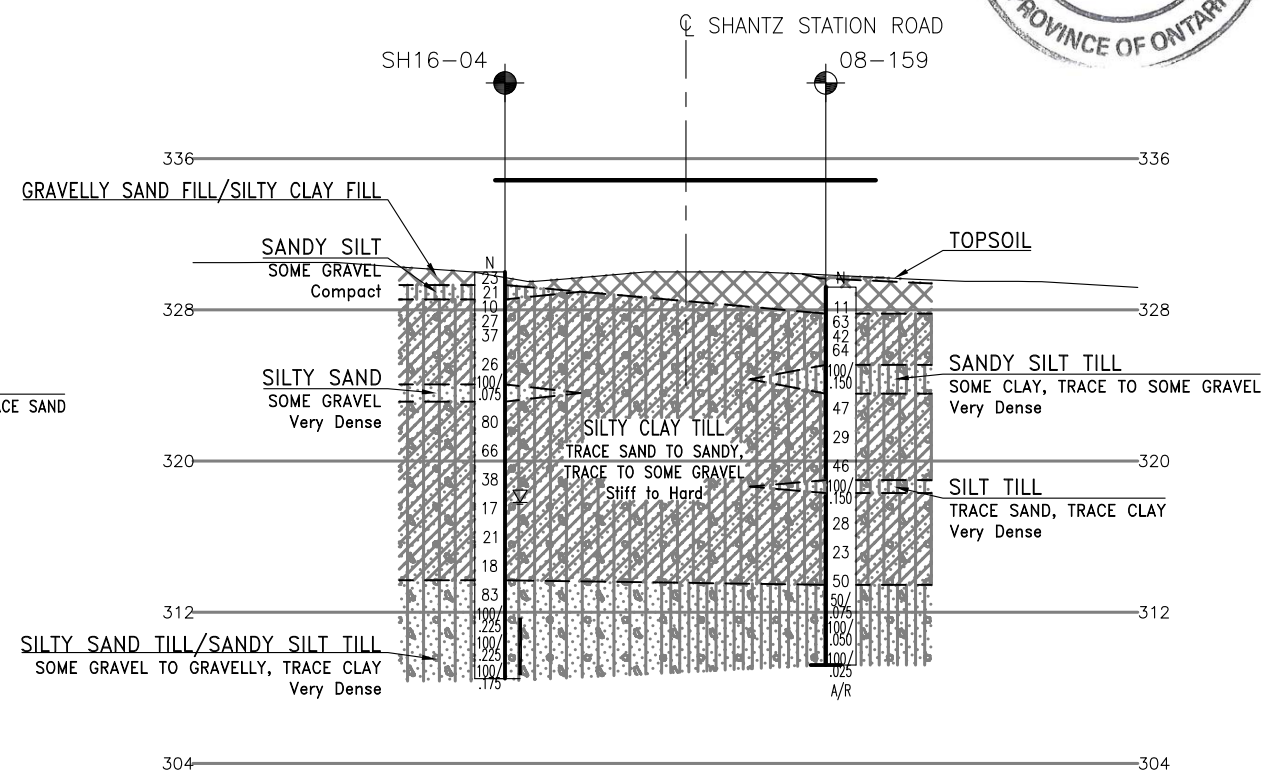
GEOCRES No. 40P9-61



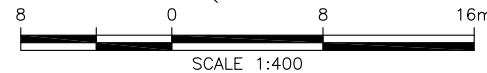
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SECTION B-B (PIER)



SECTION C-C (SOUTH ABUTMENT)

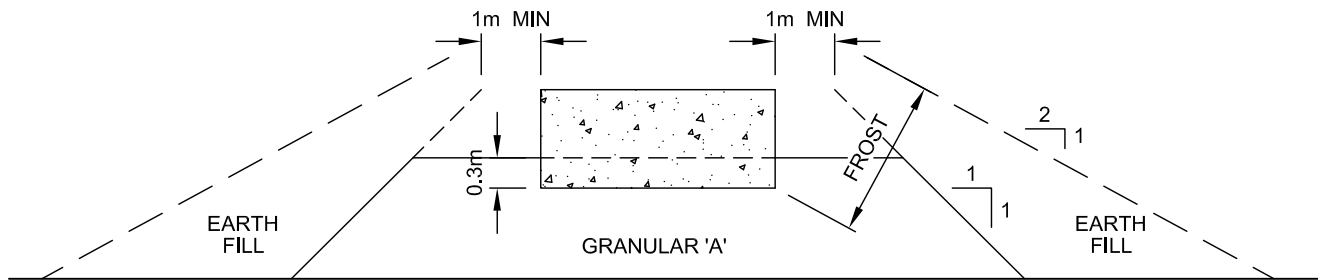


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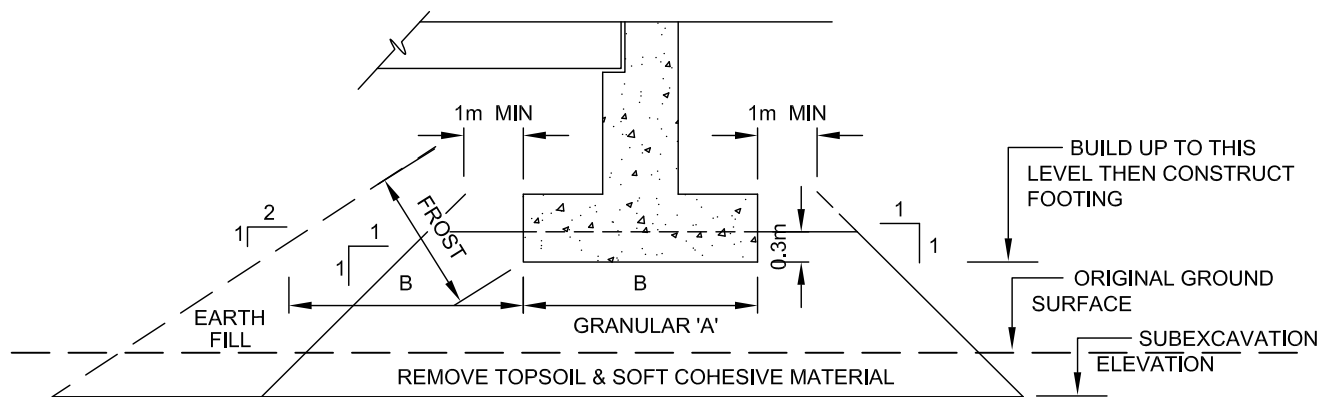


## **Appendix D**

### **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 :	SCALE :	DRAWING No.
SEPTEMBER 2016	N.T.S.	FIGURE 1



## **Appendix E**

### **Foundation Comparison**

**COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT**

Foundation Element	Spread Footings	Spread Footings on Engineered Fill	Driven Piles	Caissons
Abutments	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Generally less costly construction than deep foundation elements.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Depending of the depth of excavation, dewatering might be required prior to the excavation.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Generally less costly construction than deep foundation elements.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Better geotechnical resistance than spread footings on native.</li> <li>ii. Depending of the depth of excavation, dewatering might be required prior to the excavation.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistance may be developed by driving the piles into very dense soils and bedrock.</li> <li>ii. Comparatively short abutment stem possible</li> <li>iii. Permits integral abutment design</li> <li>iv. Installation of piles could continue in freezing weather.</li> <li>v. May require less volume of excavation than footings.</li> <li>vi. Readily installed.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher unit cost compared to footings.</li> <li>ii. When driven into hard/very dense till deposits, pipe piles are more prone to pile tip damage in comparison to H-piles.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Construction of caissons could continue in freezing weather.</li> <li>ii. High geotechnical resistance available for units founded on very dense till.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher cost than spread footings.</li> <li>ii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons under the water table.</li> <li>iii. Potential difficulty in cleaning and inspecting bases.</li> </ul>
	<b>FEASIBLE</b>	<b>FEASIBLE</b>	<b>RECOMMENDED</b>	<b>NOT RECOMMENDED</b>
Pier	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Generally less costly construction than deep foundation elements.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Depending of the depth of excavation, dewatering might be</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Generally less costly construction than deep foundation elements.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Depending of the depth of excavation,</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistance may be developed by driving the piles into very dense soils and to bedrock.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher unit cost compared to footings.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>iii. Construction of caissons could continue in freezing weather.</li> <li>iv. High geotechnical resistance available for units founded on very dense till.</li> </ul>

	<p>required prior to the excavation.</p>	<p>dewatering might be required prior to the excavation.</p>	<p>ii. When driven into hard/very dense till deposits, pipe piles are more prone to pile tip damage in comparison to H-piles.</p>	<p><b><i>Disadvantages:</i></b></p> <p>iv. Higher cost than spread footings.</p> <p>v. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons under the water table.</p> <p>vi. Potential difficulty in cleaning and inspecting bases.</p>
	<p><b>RECOMMENDED</b></p>	<p><b>FEASIBLE</b></p>	<p><b>FEASIBLE</b></p>	<p><b>NOT RECOMMENDED</b></p>

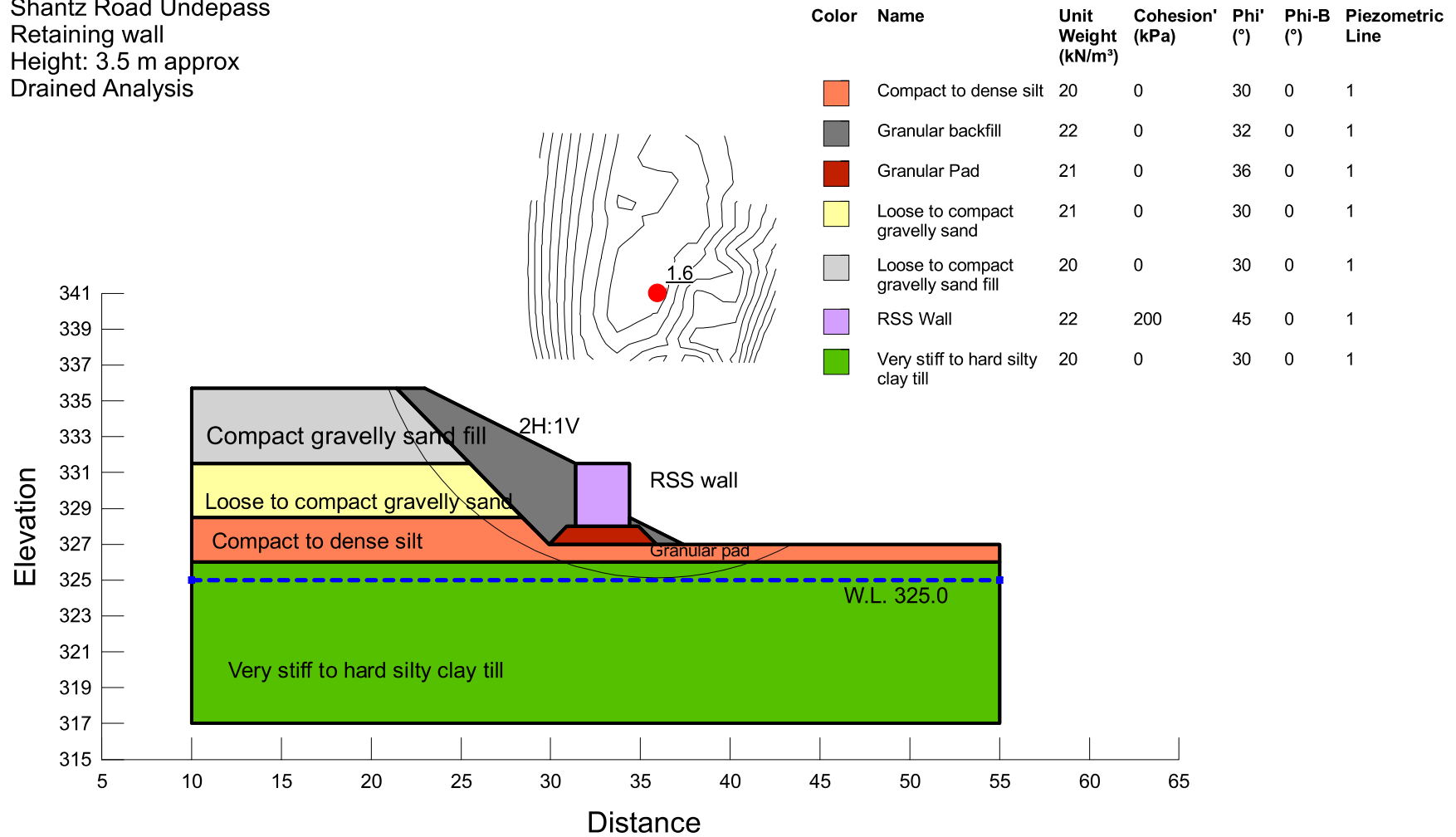




## Appendix F

### Slope Stability Output

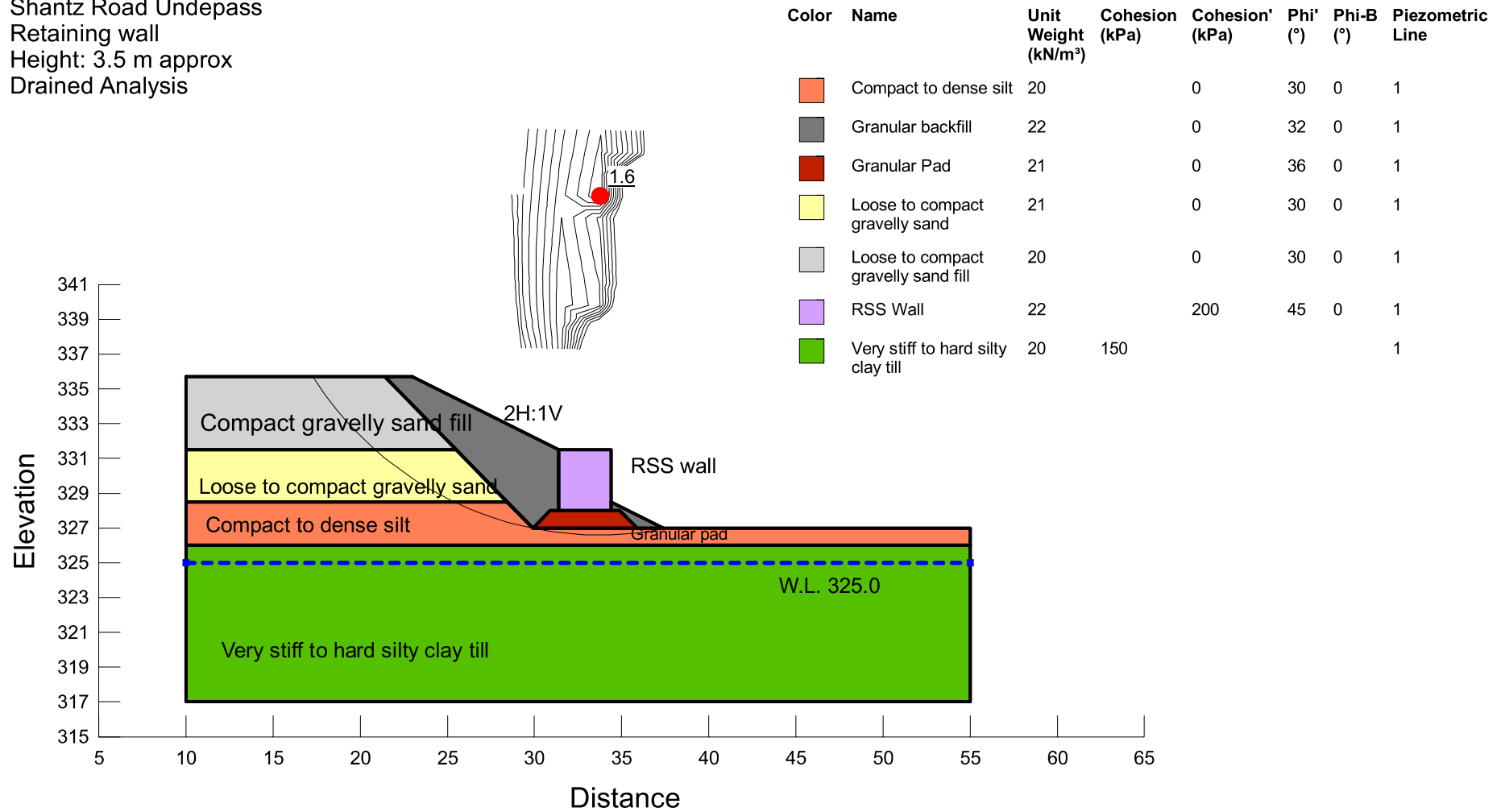
Project Number: 11375  
 Highway 7 - New  
 Shantz Road Undepass  
 Retaining wall  
 Height: 3.5 m approx  
 Drained Analysis



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 Date: 2020-05-31 ,Time: 12:51:10 PM

Figure 1F

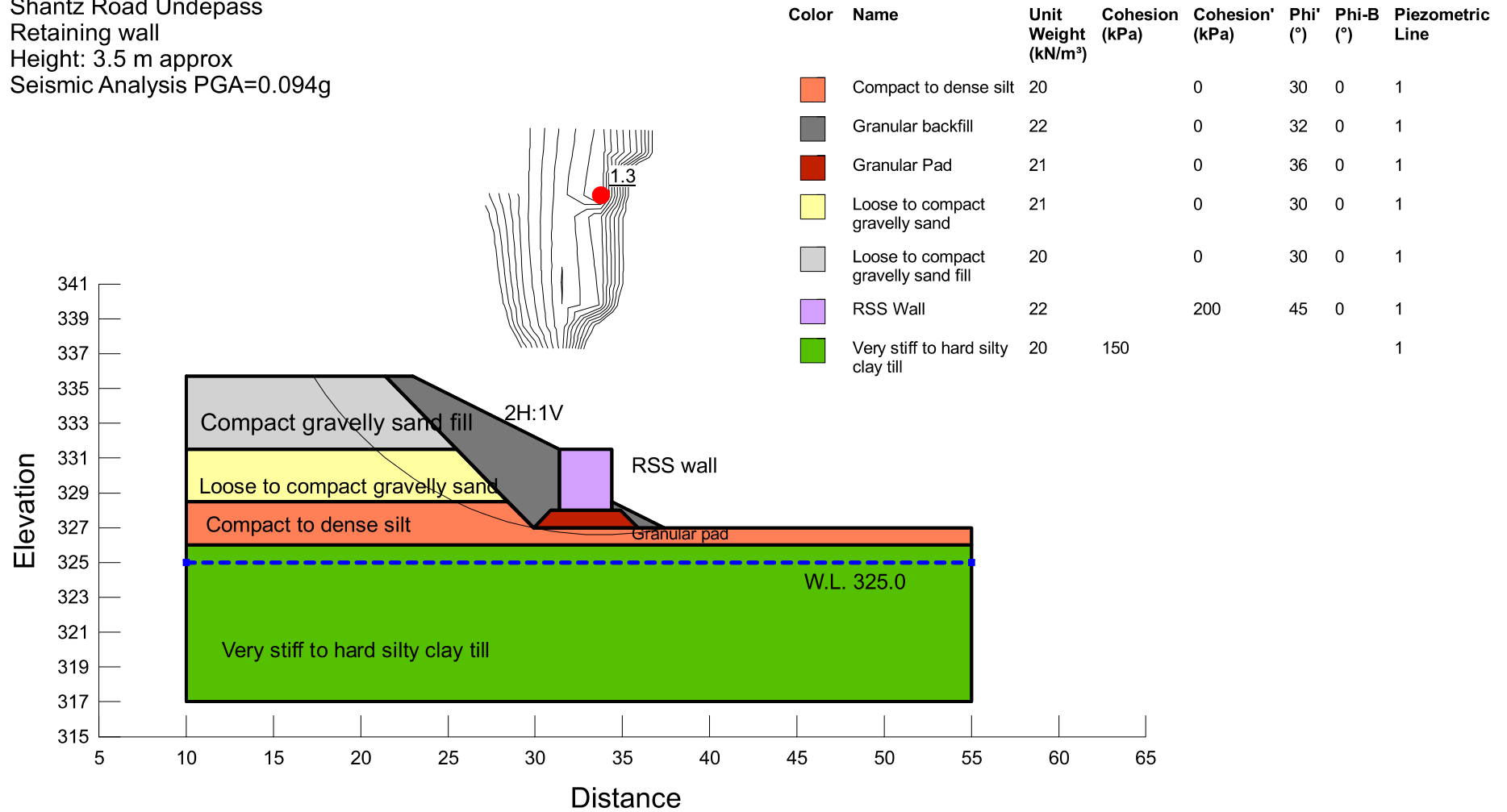
Project Number: 11375  
 Highway 7 - New  
 Shantz Road Undepass  
 Retaining wall  
 Height: 3.5 m approx  
 Drained Analysis



Directory: H:\10000+11375 Hwy 7 New PD and DD Foundations\Reports & Memos\Shantz Road\Analysis\Slope Stability\March 2020\ File Name: 11375-Shantz1\_undrained.gsz  
 Date: 2020-05-31 ,Time: 12:53:18 PM

Figure 1F

Project Number: 11375  
 Highway 7 - New  
 Shantz Road Undepass  
 Retaining wall  
 Height: 3.5 m approx  
 Seismic Analysis PGA=0.094g



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 Date: 2020-05-31 ,Time: 12:55:09 PM

Figure 3F



## **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
- SP 109S12 Amendment to OPSS 902
- OPSS PROV 903 Construction specification for deep foundations
- SP 109F57 Amendment to OPSS 903
- OPSS PROV 1010 Material specification for aggregates - base, subbase, select subgrade, and backfill material
- OPSD 3102.100 Wall abutments, backfill drain
- OPSD 3101.150 Wall Abutment, Backfill minimum granular requirement
- OPSD 3000.100 Foundation Piles, Steel H-Pile driving shoe
- OPSD 3102.100 Wall abutments, backfill drain
- OPSD 3101.150 Wall Abutment, Backfill minimum granular requirement Retainment of soils



## **2. Suggested text for a NSSP on Pile Installation**

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

The native soils at the Shantz Station Road Underpass are comprised of glacial till and are known to contain cobbles and boulders. Appropriate equipment and construction procedures will be required to penetrate or remove obstructions, such as cobbles and boulders, to permit pile installation. Pile driving must be controlled according to the criteria specified for the site.

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

## **3. Suggested text for a NSSP on Piles Driven to Bedrock**

- Piles driven to bedrock must be fitted with rock points.
- Pile driving should be terminated before the pile is damaged by overdriving.

## **4. 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. Dewatering systems must be installed and made operational prior to excavating below the groundwater level. It is also



important to minimize disturbance of the exposed sandy silt till surfaces by limiting construction traffic.