THURBER ENGINEERING LTD.

FOUNDATION INVESTIGATION AND DESIGN REPORT
GRAND RIVER BRIDGE REPLACEMENT
HIGHWAY 401
REGIONAL MUNICIPALITY OF WATERLOO, ONTARIO
G.W.P. 3080-12-00, Site No. 33-141-E/W

LATITUDE: 43.39794, LONGITUDE: -80.38619

Geocres Number: 40P8-246

Report to

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PART 1: FACTUAL INFORMATION

1.0 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted by Thurber Engineering Ltd. (Thurber) for the replacement of the Highway 401 bridge over the Grand River in the Regional Municipality of Waterloo, Ontario.

Thurber previously completed a preliminary foundation investigation and design report, dated January 18, 2018, which presented a description of the subsurface conditions anticipated at the bridge based on existing GEOCRES information, and provided preliminary geotechnical recommendations to assist selection and preliminary design of the foundation system for the replacement bridge.

The purpose of the current investigation was to explore the subsurface conditions at the proposed foundation locations, and based on the data obtained, to provide a borehole location and soil strata drawing, records of boreholes, laboratory test results, and a written description of the subsurface conditions.

Thurber completed the report as a sub-consultant to WSP who are completing the detailed design of the replacement bridge under the Ministry of Transportation Ontario (MTO) Agreement Number 3015-E-0013.

2.0 SITE DESCRIPTION

The existing Grand River Bridge consists of twin six-span structures, each with a total length of 237.1 m and width of 14.7 m. The span lengths are 39.3 m at the abutments and 39.6 m between piers. Each bridge accommodates three lanes of Highway 401 traffic. The clearance between the twin structures is 4.2 m. The bridges are supported on spread footings, and the foundation units are aligned on a 25° skew.

Road grades on Highway 401 rise from west to east on the structures, from approximate Elev. 280.3 to 287.1 on the eastbound bridge, and from approximate Elev. 280.7 to 287.4 on the westbound bridge. The Grand River channel bed is near Elev. 272.0 under the crossing. The west approach is located on an approximate 5 m high fill embankment constructed within the wide river floodplain, and the east approach is located within an approximate 8 to 9 m deep earth cut excavated into the east valley wall. The existing bridge and embankments appear to be performing well, and no evidence of slope instability was noted.

Photographs of the site are presented in Appendix A.

The site is located at the south end of Kitchener near the boundary with Cambridge to the south, and approximately 600 m west of King Street East. The adjacent lands comprise a golf course on both sides of the west approach, an established residential subdivision to the north of the east approach, and a residential subdivision under development to the south of the east approach.

The study area is located within the Waterloo Hills physiographic region, an area of sandy hills, kames, kame moraines, and ridges of sandy till, with outwash sands occupying the intervening hollows. The Grand River spillway system adjoins the hilly region, within which sand and gravelly alluvial materials are present. Bedrock lies at relatively shallow depth below the spillway (3 to 8 m at the bridge site) and consists of dolomite and shale of the Salina formation.

3.0 INVESTIGATION PROCEDURES

The site investigation was carried out during the periods of December 11 to 13, 2017, March 29 to April 30, July 23 to 30 and October 19 to 22, 2018 and comprised 22 boreholes drilled at the locations of the proposed bridge approaches and abutments. Details of the boreholes, including the total drilling depths and the length of rock coring, are presented in the following table:

Table 3.1 – Borehole Details

Foundation Unit	Borehole Designations	Borehole Depth (m)	Length of Bedrock Core (m)
West Approach	WA-04, WA-05	9.2 to 9.4	-
West Abutment	WA-01 to WA-03	11.0 to 18.0	2.8 to 9.0
Pier 1	P1-01 to P1-04	9.4 to 16.3	3.0 to 9.0
Pier 2	P2-01 to P2-04	6.9 to 14.5	3.7 to 9.5
Pier 3	P3-01 to P3-04	7.0 to 14.3	3.5 to 10.2
East Abutment	EA-01 to EA-03	14.3 to 24.8	2.3 to 6.7
East Approach	EA-04, EA-05	5.4 to 9.3	-

The approximate locations of the boreholes are shown on the Borehole Locations and Soil Strata Drawings provided in Appendix H. The latitude, longitude, and northing-easting grid co-ordinates of the boreholes are provided on the Record of Borehole sheets in Appendix B.

All borehole locations were cleared of utilities prior to commencement of drilling. The boreholes were repositioned as necessary in consideration of surface features, underground utilities, and restricted site access. The boreholes along the alignment of proposed Pier 2 were drilled from timber matting placed on the riverbed to provide a platform above the river water level Photographs 7 and 8, Appendix A).

The boreholes were advanced using solid and hollow stem augers and rotary drilling techniques within HW and NW size casing, powered by track-mounted Acker Renegade and D50 drilling equipment. Soil samples were obtained at selected intervals using a 50 mm outside diameter split-spoon sampler driven in conjunction with the Standard Penetration Test (SPT) in accordance with ASTM D 1586. Bedrock core samples were recovered using an NQ size diamond drill core barrel.

The field investigation was supervised on a full-time basis by a member of Thurber's technical staff who marked/staked the boreholes in the field, arranged for the clearance of subsurface utilities, directed the drilling, sampling and in-situ testing operations, logged the boreholes and processed the recovered soil samples for transport to Thurber's laboratory for further examination and testing. All rock cores were logged, and the Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers (19mm diameter) were installed and enclosed in filter sand in selected boreholes to permit groundwater level monitoring. The details of the piezometers and monitoring wells are shown in Table 3.2.

The boreholes in which no piezometers were installed were backfilled with bentonite and cuttings to the ground surface in general accordance with MOE Regulation 903 as amended. A slight artesian condition was observed in the monitoring well installed in Borehole P3-04; following measurement of the water level, the well was drilled out and backfilled at the source with bentonite.

Table 3.2 – Piezometer and Monitoring Well Details

Borehole	Piezometer Tip		Slotted Screen Length (m)
	Depth (m)	Elevation (m)	
WA-01	7.6	266.7	3.1
WA-03	8.9	265.5	1.5
P1-01	7.6	266.1	1.5
P1-04	6.1	267.1	1.5
P3-01	3.8	270.3	1.5
P3-04	7.7	265.9	3.1
EA-01	18.2	269.2	3.1
EA-03	15.3	272.3	3.1

4.0 LABORATORY TESTING

Routine laboratory testing was carried out at Thurber's laboratory. The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to grain size distribution analysis and Atterberg Limits testing. Point Load Testing and unconfined compressive strength tests were carried out on selected rock cores for estimating the unconfined compressive strength of the intact rock. Results of the laboratory testing are summarized on the Record of Borehole sheets in Appendix B and presented on the figures included in Appendix C.

Selected soil samples were also submitted for analytical testing to assess the potential for soil corrosion and evaluate the potential for sulphate action on concrete. The analyses were carried out by AGAT Laboratories, an independent Canadian Association for Laboratory Accreditation (CALA) accredited laboratory. The results of the analytical testing are presented in Appendix G.

5.0 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference should be made to the Record of Borehole sheets in Appendix B. Details of the encountered soil stratigraphy are presented in Appendix B and on the "Borehole Locations and Soil Strata" drawings in Appendix H. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions. It must be recognized and anticipated that soil and bedrock conditions may vary between and beyond the borehole locations.

In general, the subsurface stratigraphy encountered at the west abutment and pier locations within the river floodplain consisted of surficial topsoil and organic/alluvial layers, underlain by silty sand to sand and gravel deposits, overlying a till layer, mantling shale bedrock. At the west approach, fill was encountered above the sand and gravel in the boreholes drilled from the existing highway embankment.

At the east abutment, the boreholes were located above the valley slope (outside of the river floodplain) and the stratigraphy generally comprised a surficial pavement structure, fill and/or topsoil layer overlying native deposits of sand and clayey to sandy silt till, underlain by bedrock.

More detailed descriptions of the individual strata are presented below.

5.1 Pavement Structure

A pavement structure comprising 200 to 350 mm of asphalt over sand and gravel was encountered in Boreholes WA-04, WA-05, EA-04 and EA-05 drilled on the paved shoulder of Highway 401 at the west and east approaches. The pavement granular material extended to depths of 2.2 m at the west approach and 1.2 to 1.4 m at the east approach.

SPT ‘N’ values recorded in the granular material varied from 13 blows per 0.3 m penetration to 100 blows per 0.275 m, indicating a compact to very dense condition. Measured moisture contents ranged from 2 to 12%.

The results of grain size distribution tests carried out on the sand and gravel fill are shown on Figure C1 in Appendix C and summarized below:

Soil Particle	Percentage (%)
Gravel	26 to 37
Sand	49 to 62
Silt + Clay	12 to 14

5.2 Fill

Silty sand embankment fill was encountered below the pavement structure in Boreholes WA-04 and WA-05 drilled at the west approach. The base of the embankment fill was encountered at 7.2 m depth (Elev. 272.8) in both boreholes.

A 1.0 m thick layer of sand fill was encountered at the ground surface in Borehole EA-02 drilled adjacent to the front face of the existing east abutment. In addition, sand and gravel fill was encountered to a depth of 5.6 m in Borehole EA-03 drilled behind the south end of the abutment. The lower boundary of the fill in these boreholes was at Elev. 281.8 and 282.0.

SPT ‘N’ values recorded in the fill varied widely from 5 blows per 0.3 m penetration to 100 blows per 0.125 m, indicating a loose to very dense condition. Measured moisture contents in the fill ranged from 2 to 28%, typically 2 to 9%.

The results of grain size distribution tests carried out on the sand and gravel fill are included on Figure C1 in Appendix C and summarized below:

Soil Particle	Percentage (%)
Gravel	38 to 46
Sand	46 to 47
Silt + Clay	8 to 15

The results of grain size analyses carried out on samples of the silty sand fill are presented on Figure C2 of Appendix C, and the results were as follows:

Soil Particle	Percentage (%)
Gravel	17 to 18
Sand	48 to 49
Silt	25 to 26
Clay	8 to 9

5.3 Topsoil and Organic Deposits

A layer of topsoil or organic material was encountered at the ground surface in selected boreholes at the abutments and Piers 1 and 3. The thickness of the organic layers were as follows:

- A 150 to 200 mm thick topsoil layer was reported in Boreholes WA-01 to WA-03 at the west abutment;
- A layer described as organics/topsoil was encountered in Boreholes P1-01 to P1-04 at Pier 1. This layer ranged from 275 to 1200 mm in thickness;
- A 75 to 250 mm thick layer of organics/topsoil was encountered in Boreholes P3-01, P3-02 and P3-04 at Pier 3. In Borehole P3-02, this layer was underlain by an 850 mm thick layer of silty sand with organics, and by a further 300 mm thick layer of peat with a lower boundary at 1.4 m depth (Elev. 272.2);

- A 300 mm thick layer of topsoil fill was encountered at the ground surface in Boreholes EA-01 and EA-03 at the east abutment.

Moisture contents of 12 to 37% were recorded in the organic deposits.

The topsoil thickness may vary at locations between and beyond the boreholes, and the recorded thicknesses are not intended for use in estimating quantities.

5.4 Silty Clay Till

A thin layer of silty clay till was encountered underlying the pavement structure in Borehole EA-04 and below a sand layer (described below) in Borehole EA-01. The clay till layer was 0.8 m thick with a lower boundary at 2.2 m depth (Elev. 285.1 and 285.9).

SPT ‘N’ values of 32 and 36 blows per 0.3 m were recorded within the till, indicating a hard consistency. Moisture contents of 11 and 20% were measured in the till.

The results of a grain size distribution analysis carried out on one sample of the clay till are shown on Figure C3 in Appendix C. The results indicated 6% gravel, 10% sand, 37% silt, and 47% clay.

Atterberg limits testing was carried out on one sample of the till. The measured liquid limit, plastic limit and plasticity index were 33, 16 and 17, respectively. These results, which are plotted on Figure C11 in Appendix C, indicate that the sample tested consists of silty clay of low plasticity (CL).

Till soils frequently contain cobbles and boulders, and these should be anticipated when excavating during construction.

5.5 Sand

A sand stratum was encountered below the fill and clay till at depths of 1.2 to 2.2 m (Elev. 286.7 to 285.1) in Boreholes EA-01, EA-04 and EA-05 drilled above the east valley slope. The sand deposit was 3.4 to 5.0 m thick and extended to at depths of 5.6 and 7.2 m (Elev. 281.7 and 280.9) in Boreholes EA-01 and EA-04, respectively. Borehole EA-05 was terminated in the sand at 5.4 m depth (Elev. 282.6).

SPT ‘N’ values recorded in the sand deposit ranged from 47 blows per 0.3 m to 100 blows per 0.025 m, indicating a generally very dense relative density. Measured moisture contents in the sand generally ranged from 2 to 7%, locally 12% near the base of this unit in Borehole EA-04.

The results of grain size distribution analyses carried out on selected samples of the sand are shown on Figure C4 in Appendix C, and the results were as follows:

Soil Particle	Percentage (%)
Gravel	0 to 29
Sand	56 to 86
Silt + Clay	14 to 26

5.6 Silty Clay to Clayey Silt Till

A till deposit consisting of silty clay to clayey silt with trace sand to sandy was encountered below the fill and sand layers in Boreholes EA-01 to EA-04 drilled at the east abutment. The cohesive till deposit extended to depths of 7.6 to 14.0 m (Elev. 277.0 to 273.4) in Boreholes EA-01 to EA-03, indicating a deposit thickness of 5.1 to 8.4 m. Borehole EA-04 was terminated in the clayey silt till at 9.3 m depth (Elev. 278.8).

The SPT ‘N’ values recorded within the cohesive till ranged from 62 blows per 0.3 m to 100 blows per 0.1 m penetration, indicating a hard consistency. The measured moisture contents ranged from 7 to 15%.

The results of grain size distribution analyses carried out on samples of the silty clay to clayey silt till are shown on Figure C5 included in Appendix C, and the results were as follows

Soil Particle	Percentage (%)
Gravel	0 to 4
Sand	9 to 33
Silt	44 to 72
Clay	16 to 28

Atterberg limits testing was carried out on one sample of the till. The measured liquid limit, plastic limit and plasticity index were 17, 11 and 6, respectively. These results, which are plotted on Figure C11 in Appendix C, indicate that the sample tested consists of low plastic silty clay to clayey silt (CL-ML).

Till soils frequently contain cobbles and boulders, and these should be anticipated when excavating during construction.

5.7 Sand and Silt Till

The till deposit encountered in Boreholes EA-01 to EA-03 graded to sand and silt with trace to some clay at depths of 7.6 to 14.0 m (Elev. 277.0 to 273.4). The cohesionless till extended to bedrock contacted at depths of 12.0 to 18.2 m (Elev. 270.8 to 269.1).

The SPT 'N' values recorded within the cohesionless till ranged from 74 blows per 0.3 m to 100 blows per 0.025 m penetration, indicating a very dense condition. The measured moisture contents ranged from 5 to 10%.

The results of grain size distribution analyses carried out on samples of the sand and silt till are shown on Figure C6 included in Appendix C, and the results were as follows

Soil Particle	Percentage (%)
Gravel	3 to 9
Sand	37 to 45
Silt	42 to 45
Clay	9 to 10

Locally in Borehole EA-03, a sand layer of undefined thickness was encountered within the till in the sample recovered from 13.7 m depth (Elev. 273.9). The results of a grain size distribution analysis carried out on the sand are included on Figure C4 in Appendix C.

The till deposit contains cobbles and boulders, and these should be anticipated when excavating during construction.

5.8 Silty to Gravelly Sand

Silty to gravelly sand was encountered underlying the topsoil/organic layers or riverbed in Boreholes WA-03, P1-04, P2-03, P2-04, P3-01, P3-02 and P3-04. The sand layer was 1.4 to 5.5 m thick, with a lower boundary at depths of 1.4 to 5.7 m (Elev. 271.9 to 268.2). This material may represent backfill material to the existing bridge foundations.

The SPT 'N' values recorded within the sand varied widely from 3 blows per 0.3 m to 100 blows per 0.125 m penetration, indicating a very loose to very dense condition. The high 'N' values may reflect the presence of cobbles and boulders in the sand. The measured moisture contents typically ranged from 10 to 20%, locally from 5 to 37%.

The results of grain size distribution analyses carried out on samples of the sand are shown on Figure C7 included in Appendix C, and the results were as follows

Soil Particle	Percentage (%)
Gravel	17 to 29
Sand	43 to 60
Silt	17 to 23
Clay	5 to 10

The silty to gravelly sands contain cobbles and boulders, and these should be anticipated when excavating during construction.

5.9 Sand and Gravel

A cohesionless deposit primarily consisting of sand and gravel with cobbles and boulders was encountered on the riverbed at Boreholes P2-01 to P2-03, and below the fill and topsoil/organic layers in Boreholes WA-01, WA-02, WA-04, WA-05, P1-01 to P1-03, and P3-03 drilled within the floodplain. Where fully penetrated the thickness of the sand and gravel layer ranged from 2.3 to 4.9 m, locally 7.0 m. The lower boundary was encountered at depths of 2.3 to 7.3 m (Elev. 270.0 to 266.2). Boreholes WA-04 and WA-05 drilled within the west approach embankment were terminated within the stratum at 9.2 and 9.4 m depth (Elev. 270.8 and 270.6). The sand and gravel may locally be a backfill material related to previous foundation construction.

SPT ‘N’ values recorded in the sand and gravel deposit varied widely and ranged from 1 blows per 0.3 m penetration to 100 blows per 0.025 m, indicating a very loose to very dense relative density. Measured moisture contents in the sand and gravel ranged from 3 to 22%, typically about 7 to 20%.

The results of grain size distribution analyses carried out on samples of the sand and gravel are shown on Figures C8 and C9 included in Appendix C, and the results were as follows

Soil Particle	Percentage (%)
Gravel	29 to 65
Sand	24 to 59
Silt + Clay	1 to 23

Cobbles and boulders and occasional organics were noted within the sand and gravel deposit.

5.10 Silty Sand to Sandy Silt Till

A till layer comprising silty sand to sandy silt was encountered underlying the topsoil/organic and sand and gravel layers in Boreholes WA-01 to WA-03 and in Boreholes P1-01 to P1-04, P2-01 to P2-04, and P3-01 to P3-04 at the piers. The cohesionless till layer was 0.9 to 4.8 m thick, and was underlain by bedrock at depths of 3.0 to 9.1 m (Elev. 270.6 to 265.1).

The SPT 'N' values recorded within the till generally ranged from 28 blows per 0.3 m to 100 blows per 0.1 m penetration, indicating a compact to very dense condition, typically very dense. The measured moisture contents typically ranged from 5 to 11%.

The results of grain size distribution analyses carried out on samples of the sand till are shown on Figures C10 included in Appendix C, and the results were as follows:

Soil Particle	Percentage (%)
Gravel	0 to 15
Sand	37 to 54
Silt	25 to 47
Clay	8 to 16

The till soils contain cobbles and boulders, and these should be anticipated when excavating during construction.

5.11 Bedrock

Bedrock was encountered underlying the overburden soils in all boreholes, with the exception of Boreholes WA-04, WA-05, EA-04 and EA-05 drilled at the approaches. The depths and elevations at which bedrock was encountered are summarized in Table 5.1.

The bedrock was proven by coring to a depth of 2.3 to 10.2 m below the interpreted bedrock surface. As the bedrock was highly weathered and the overlying till deposits contain cobbles and boulders, coring operations were sometimes commenced in bouldery material above the bedrock surface or after augering a short distance into the bedrock, and interpretation of the approximate bedrock surface based on the core recovery was necessary.

Photographs of the recovered rock cores are presented in Appendix D.

Table 5.1 – Bedrock Contact Depths and Elevations

Foundation Element	Borehole	Bedrock Surface	
		Depth (m)	Elevation
West Abutment	WA-01	9.1	265.1
	WA-02	8.2	265.6
	WA-03	7.6	266.8
Pier 1	P1-01	7.6	266.0
	P1-02	7.3	266.2
	P1-03	6.4	266.5
	P1-04	6.2	267.0
Pier 2	P2-01	5.3	266.7
	P2-02	4.9	267.1
	P2-03	4.7	267.3
	P2-04	3.0	268.9
Pier 3	P3-01	3.5	270.6
	P3-02	3.6	270.0
	P3-03	2.7	269.8
	P3-04	3.0	270.6
East Abutment	EA-01	18.2	269.1
	EA-02	12.0	270.8
	EA-03	17.8	269.8

The bedrock was visually identified as grey shale interbedded with carbonates (limestone and dolostone). In general, the upper 3.5 to 5.5 m of bedrock is highly weathered, with some shale beds completely weathered to resemble a hard clay. Vugs were observed in selected cores of limestone and dolostone and seams of gypsum and quartz were noted in cores. Total core recovery varied widely from 20% to 100%. The RQD of the rock cores ranged from 0 to 100%. In general, RQD was determined to be very poor to fair quality (<50%) in the upper 3.0 to 4.5 m, generally becoming fair to very good (>50%) with depth.

The unconfined compressive strength (UCS) of the rock, estimated from the results of point load tests, typically varied from 5 to 250 MPa, indicating a very weak to very strong rock strength classification with localized results above 250 MPa, indicating extremely strong rock. In general, the strength of the shale typically ranged from 5 to 50 MPa and the carbonates (limestone/dolostone) ranged from about 50 to 250 MPa. The estimated rock strength values are based on correlation with point load test results that were conducted on selected intact rock cores recovered from the boreholes. The point load test results are provided in Appendix C.

Uniaxial compressive strength testing was carried out on eight samples of the bedrock, and the results are included in Appendix C. The measured compressive strengths ranged from 30 to 163 MPa, indicating a medium strong to very strong strength classification.

5.12 Groundwater Conditions

The rotary borehole advancement methodology used during the investigation requires maintaining a head of water in order to stabilize the base. Further, rock coring operations introduce water into the boreholes. In this regard, water levels observed at the termination of boreholes drilled with these methodologies may not accurately represent the long-term stabilized ground water level and are not reported here. The water levels measured upon completion of drilling in the boreholes augered at the approaches are summarized in Table 5.2.

Table 5.2 - Water Level Observations in Approach Boreholes

Borehole Number	Date	Observed Water Level on Completion		Remark
		Depth (m)	Elevation (m)	
WA-04	April 30, 2018	7.5	272.5	Upon completion
WA-05	December 11, 2017	7.3	272.7	Upon completion
EA-04	April 30, 2018	Dry	-	Upon completion
EA-05	December 11, 2017	Dry	-	Upon completion

The above water level measurements are short-term observations and seasonal fluctuations of the groundwater level are to be expected.

The groundwater conditions recorded in the piezometers installed in selected boreholes are summarized in Table 5.3.

Table 5.3 - Groundwater Observations in Piezometers

Foundation Element	Borehole	Date	Approximate Water Level (m)		Comment
			Depth	Elevation	
West Abutment	WA-01	April 13, 2018 August 9, 2018 October 22, 2018	1.7 2.0 1.9	272.6 272.3 272.4	
	WA-03	April 2, 2018 April 3, 2018 April 5, 2018 April 9, 2018 April 11, 2018 August 9, 2018 October 22, 2018	1.0 1.9 1.6 1.7 1.8 2.3 2.1	273.4 272.5 272.8 272.7 272.6 272.1 272.3	
Pier 1	P1-01	April 5, 2018 April 9, 2018 April 11, 2018 October 22, 2018	0.8 1.0 1.1 1.3	272.9 272.7 272.6 272.4	
	P1-04	April 10, 2018 April 11, 2018 October 22, 2018	0.6 0.8 1.0	272.6 272.4 272.2	
Pier 3	P3-01	August 9, 2018 October 22, 2018	1.0 1.1	273.1 273.0	
	P3-04	October 22, 2018	1.0 ags	274.6	Artesian
East Abutment	EA-01	April 27, 2018 August 9, 2018 October 22, 2018	6.5 6.0 6.0	280.9 281.4 281.4	
	EA-03	August 9, 2018 October 22, 2018	8.4 9.0	279.2 278.6	

* ags = above ground surface

A water depth of 0.3 m was measured at the locations of Boreholes P2-01 to P2-04 drilled within the river.

In general, the water level in the floodplain is expected to be governed by the prevailing water level in the river. A normal river water level of Elevation 272.4 m (September 2016) is shown on the preliminary General Arrangement drawing for the bridge.

Seasonal fluctuations of the river and groundwater level should be expected. In particular, the groundwater and river water levels may be at a higher elevation after periods of significant or prolonged precipitation, or after snowmelt.

5.13 Corrosivity and Sulphate Test Results

Samples of the fill and native soils were submitted for analytical testing of corrosivity parameters and sulphate. The results of the analytical tests are shown in Table 5.4. The laboratory certificates of analysis are presented in Appendix G.

Table 5.4 – Analytical Test Results

BH No. (Sample ID)	Depth (m)	Description	Sulphide (%)	Chloride (µg/g)	Sulphate (µg/g)	pH	Electrical Conductivity (mS/cm)	Resistivity (ohm.cm)	Redox Potential (mV)
WA-02	1.5-2.1	Sand and Gravel	0.07	42	256	7.79	0.464	2160	178
WA-04 (B2)	1.5-2.1	Sand and Gravel Fill	<0.05	481	26	9.93	1.25	800	173
P1-02	0.8-1.4	Sand	<0.05	188	35	8.86	0.377	2650	194
P2-02	2.3-2.9	Sand and Gravel	<0.05	26	23	9.13	0.136	7350	196

6.0 MISCELLANEOUS

Thurber Engineering positioned the boreholes in the field using a hand-held GPS unit, with consideration of site features and access limitations. The co-ordinates and ground elevations at the borehole locations, with the exception of Boreholes WA-04 and WA-05, were subsequently determined by WSP surveyors using a total station with an accuracy of about 2 mm.

Walker Drilling of Utopia, Ontario supplied and operated the drilling and sampling equipment for the field program.

Full time supervision of the field activities, including obtaining utility clearances, was carried out by Ms. Judy Mei, Mr. Amir Fereidouni, Mr. Sam Bastan, Mr. Bryan Lui and Mr. Stephen Jones of Thurber Engineering. Overall supervision of the field program was performed by Mr. Stephane Loranger and Mr. Murray Anderson, P.Eng. of Thurber.

Interpretation of the field data and preparation of the report were performed by Mr. Karel Furbacher, P.Eng. and Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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FOUNDATION INVESTIGATION AND DESIGN REPORT
GRAND RIVER BRIDGE REPLACEMENT
HIGHWAY 401
REGIONAL MUNICIPALITY OF WATERLOO, ONTARIO

Geocres Number: 40P8-246

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7.0 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations for design of the foundation system for the replacement of the Grand River Bridge carrying Highway 401.

Replacement of the existing six-span twin bridge structure with a new twin four-span structure is proposed. The proposed bridge will have a total length of 225 m between abutments, approximately 12 m shorter than the existing structure, and the new abutments will be placed inside of the existing abutments. The span lengths for the new bridge will range from 40.0 to 75.0 m.

The width of the structures will be increased from 14.7 m each (three lanes) to 30.3 m for the westbound structure and 23.3 m for the eastbound structure. The clearance between structures will be 2.0 m. The existing 25° skew of the abutments and piers will be maintained. Proposed road grades on Highway 401 will be near existing grades.

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

The interpretation and recommendations are intended for the use of the design consultant and the Ministry of Transportation (MTO) and shall not be relied upon by any other parties including the construction contractor, or used for any purposes other than development of the project design. Comments on construction methodology and equipment, where presented, are provided only to highlight those aspects that could affect the design of the project. Contractors must make their own assessment of the factual information presented in Part 1 of the report, and the implications on equipment selection, construction methodology, and scheduling.

The report references the Canadian Highway Bridge Design Code published in December 2014 (CHBDC 2014) by the CSA Group. In accordance with the CHBDC 2014, a consequence classification of “typical consequence” and a degree of site and prediction model understanding of “typical understanding” have been assumed.

8.0 FOUNDATION DESIGN

In general, the subsurface stratigraphy encountered at the west abutment and pier locations within the river floodplain consisted of surficial topsoil and organic/alluvial layers, underlain by silty sand to sand and gravel deposits containing cobbles and boulders, overlying a till layer, mantling shale bedrock. At the west approach, fill was encountered above the sand and gravel in the boreholes drilled from the existing highway embankment.

Groundwater was measured at depths ranging from 0.6 to 2.3 m (Elev. 273.1 to 272.1) in piezometers installed at the proposed west abutment, Pier 1 and Pier 3 within the floodplain. Artesian head rising to about 1.0 m above the ground surface was observed in one piezometer installed at Pier 3. A river water depth of 0.3 m was measured at the boreholes drilled near proposed Pier 2. In general, the water level in the floodplain is expected to be governed by the prevailing water level in the river.

At the east abutment, the boreholes were located above the valley slope (outside of the river floodplain) and the stratigraphy generally comprised a surficial pavement structure, fill and/or topsoil layer overlying native deposits of sand and clayey to sandy silt till, underlain by bedrock. Groundwater was measured at depths of 6.0 to 9.0 m (Elev. 281.4 to 278.6).

Based on the subsurface conditions at the site, consideration was given to supporting the bridge using the following foundation types:

- Spread footings on native soil or bedrock
- Drilled shafts (caissons)
- Driven steel H-piles
- Socketed H-Piles
- Drilled-in Pipe Piles
- Micropiles

A comparison of the technical advantages and disadvantages of the alternative foundation schemes is presented in Appendix E. Recommendations for feasible foundation alternatives are presented in the following sections. A foundation scheme preferred from a foundations perspective is then recommended.

8.1 Spread Footings on Native Soil

The borehole information indicates that the relative density of the silty to gravelly sands and sand and gravel deposits present in the floodplain varies widely from very loose to very dense. The variation may have resulted from disturbance during construction of the existing bridge foundations, and portions of these materials may comprise foundation backfill placed without engineered control. The underlying silty sand till is typically very dense, however this stratum was not identified in all boreholes at the piers. Therefore supporting the piers and west abutment on spread footings constructed on the river bed and floodplain deposits is not recommended and this option has not been developed further.

The use of spread footings founded on native soil could be considered for the east abutment. Based on the preliminary General Arrangement drawing, the design founding level for spread footings is expected to be at or below Elevation 280. The borehole information (Boreholes EA-01 to EA-03) indicates that the soil at this elevation consists of hard native silty clay to clayey silt till. Spread footings founded on the hard native soil at or below Elevation 280 may be designed using factored geotechnical resistances of 600 kPa and 400 kPa at factored ULS and SLS, respectively.

The resistance values are for a minimum 2 m wide footing subjected to vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance values used in design must be reduced in accordance with the CHBDC (2014) Clauses 6.10.2 to 6.10.4.

The geotechnical resistances at SLS are based on an estimated settlement not exceeding 25 mm. This settlement should be essentially complete by the end of construction.

The lateral resistance developed along the base of concrete footings founded on the hard till may be computed using an ultimate friction coefficient of 0.45. A geotechnical resistance factor of 0.8 should be applied to this ultimate value.

Footing construction should be in accordance with OPSS.PROV 902. Founding surfaces should be protected from disturbance during construction. The exposed surface should be protected from deterioration by placing a minimum 75 mm thick working mat of concrete immediately following approval of the founding surface.

The overall inclination of the existing valley slope at the east abutment generally varies from about 2.7H:1V to 4.0H:1V. It is recommended that the abutment footings be placed a minimum distance of twice the footing width from the face of the slope to achieve the design resistance values and minimize the potential for impact on the stability of the slope. The toe of slope should be protected from river erosion and potential future regression of the slope, or the footing setback should be increased in anticipation of future erosion. Assessment of morphological changes in the river course and required scour protection measures should be carried out by a qualified and experienced river and/or hydraulic engineering specialist.

8.2 Spread Footings on Bedrock

Consideration may be given to supporting the piers and west abutment on spread footings extended down to bear on the underlying bedrock. The archive drawings indicate that the existing piers and west abutment are supported by footings founded below the bedrock surface, suggesting that this may be a feasible alternative for the new footings. Extending footings down to bedrock is not warranted at the east abutment in view of the competency of the native soils and the significant depth to bedrock.

It must be noted that excavation for footing construction will require cofferdam installation and a significant dewatering effort. Installation of sheet pile cofferdams may be problematic in view of the relative density of the riverbed deposits and the presence of cobbles and boulders. Dewatering equipment may need to handle significant flow volumes in view of the permeable nature of the weathered bedrock and overlying granular soils, and tremie methods may be required to place concrete if dewatering proves impractical or is not approved by permitting authorities. The presence of the existing foundations may also impact construction and staging of new foundations. For these reasons, spread footings on bedrock are not the preferred foundation type for the bridge replacement.

The upper 3.5 to 5.5 m of the bedrock at the site is highly weathered, of poor quality, and contains gypsum layers and solution cavities. In consideration of the poor quality of the bedrock, it is recommended that footings be founded at least 1.2 m below the interpreted surface of the weathered bedrock. Based on the borehole data and a 1.2 m embedment depth into the bedrock, the design founding elevation will vary as summarized in Table 8.1.

Spread footings founded at least 1.2 m below the bedrock surface may be designed using geotechnical resistance values of 750 kPa and 500 kPa at factored ULS and factored SLS, respectively.

Table 8.1 – Recommended Founding Elevation in Weathered Bedrock

Foundation Unit	Reference Boreholes	Recommended Founding Elevation
West Abutment	WA-01 to WA-03	263.9 to 265.6
Pier 1	P1-01 to P1-04	264.8 to 265.8
Pier 2	P2-01 to P2-04	265.5 to 267.7
Pier 3	P3-01 to P3-04	268.6 to 269.4

Extending footings down a further 2.3 to 4.3 m into the bedrock to achieve higher capacities on the underlying sound bedrock is not considered to be practical in view of the excavation depth and dewatering requirements.

The recommended geotechnical resistance is based on a footing subjected to vertical concentric loading. Where eccentric or inclined loads are applied, the resistance values used in design must be reduced in accordance with the CHBDC (2014) Clause 6.10.3 and Clause 6.10.4.

The lateral resistance developed along the base of concrete footings founded on the weathered bedrock may be computed using an ultimate friction coefficient of 0.55. A geotechnical resistance factor of 0.8 should be applied to this ultimate value.

8.3 Drilled Shafts (Caissons)

The piers and west abutment may be supported on caissons socketed into bedrock. The upper 3.5 to 5.5 m of the bedrock is highly weathered, of poor quality and contains solution cavities, and therefore it is recommended that the caissons be socketed into the underlying less weathered bedrock of fair to very good quality without voids, clay seams and highly fractured zones. The use of caissons may also be considered for support of the east abutment, either founded in the very dense till or socketed into bedrock.

The caissons will develop axial resistance through a combination of sidewall shear and end bearing in the rock socket. The axial geotechnical resistances at ULS recommended for design of caissons with socket lengths of at least two times the diameter, 5.0 m, and 8.0 m into the less weathered bedrock are presented in Table 8.2. The SLS resistance will not govern design.

The contribution of shaft and end-bearing resistance to the computed axial resistance was based on a factored shaft resistance along the rock socket sidewall of 350 kPa and a factored base resistance of 2,500 kPa. The resistance values assume that the socket sidewalls and base will not be softened, smeared or fractured by drilling methods.

Table 8.2 – Recommended Axial Geotechnical Resistances for Caisson Design

Caisson Diameter (m)	Socket Length in Less Weathered Bedrock (m)	Factored Axial Geotechnical Resistance at ULS (kN)
0.9	1.8	3,300
	5.0	6,500
	8.0	9,500
1.2	2.4	6,000
	5.0	9,400
	8.0	13,400
1.5	3.0	9,300
	5.0	12,600
	8.0	17,600
1.8	3.6	13,500
	5.0	16,200
	8.0	22,200
2.1	4.2	18,300
	5.0	20,000
	8.0	27,000

The interpreted top of the less weathered bedrock to be used in determining the base elevations of the caisson sockets is summarized in Table 8.3.

Table 8.3 – Interpreted Elevation of Less Weathered Bedrock

Foundation Unit	Reference Boreholes	Interpreted Top of Less Weathered Bedrock (Elev.)	
		North End	South End
West Abutment	WA-01 to WA-03	261.0	262.5
Pier 1	P1-01 to P1-04	261.5	261.0
Pier 2	P2-01 to P2-04	262.0	262.0
Pier 3	P3-01 to P3-04	265.0	265.0
East Abutment	EA-01 to EA-03	264.5	264.5

Uplift forces on the foundations will be resisted by shaft resistance developed along the sidewalls of the caisson socket in shale. For uplift resistance, the factored shaft resistance at ULS may be taken as 75% of the shaft resistance value indicated above for axial compressive loads. SLS conditions will not apply.

Based on the borehole data, caisson excavation will extend through cohesionless sand and gravel deposits with cobbles and boulders, till materials containing cobbles and boulders, and highly weathered bedrock containing voids, clay seams and highly fractured zones, prior to encountering less weathered bedrock. The bedrock consists of shale with dolostone and limestone layers. Augering and socketing operations may be difficult and significantly impacted by these conditions. The Contractor must be prepared to penetrate these materials to advance the caisson into bedrock and form the rock socket. It is anticipated that a permanent caisson liner will be required to support the excavation sidewalls in the cohesionless overburden soils and upper highly weathered portion of the bedrock, and prevent materials from falling into the socket.

High volumes of seepage should be anticipated into caisson excavations socketed into bedrock, and measures such as heavy duty pumping to maintain a dry excavation and enable concrete placement in a dewatered condition may not be practical. It is anticipated that placement of concrete using tremie methods will be required.

After each rock socket is drilled, cleaned and approved, structural concrete must be placed within 6 hours to prevent softening of the shale exposed on the base and sidewalls of the excavation.

At the east abutment, caissons may alternatively be founded in the hard to very dense till deposits above the bedrock surface. The axial geotechnical resistances at ULS recommended for design of caissons with a selected length of 8.0 m below the base of the pile cap (assumed at Elev. 280.0) are presented in Table 8.4. The factored SLS resistances for 10 mm of settlement are also provided.

Table 8.4 – Recommended Axial Resistances for Caissons in Till

Caisson Length (m)	Base Elevation	Caisson Diameter (m)	Factored Axial Resistance at ULS (kN)	Factored Axial Resistance at SLS (kN)
8.0	272.0	0.9	2,000	1,600
		1.2	3,500	2,100
		1.5	5,000	2,600
		1.8	7,000	3,200
		2.1	9,000	3,700

The axial loading required to achieve 25 mm of vertical deflection will exceed the factored resistance at ULS. The factored SLS resistance for 25 mm of settlement will therefore not govern design.

A minimum centre-to-centre spacing of two caisson diameters should be maintained between caissons.

Caisson excavation at the east abutment will extend through hard, cohesive silty clay to clayey silt till and very dense sand and silt till above the bedrock. Till deposits inherently contain cobbles and boulders and these may be encountered during excavation. The installation (augering) equipment must be capable of penetrating hard glacial till, and dislodging, removing or penetrating any obstructions such as cobbles, boulders, and rock fragments.

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

Caisson installation must be in accordance with OPSS.PROV 903.

Selection of the methods and equipment employed to install the caissons is the responsibility of the Contractor. However, the contract documents should contain a statement to alert bidders of the potential issues outlined above. The wording for an NSSP to be included in the tender documents is provided in Appendix F.

8.4 Driven Steel H-Pile Foundations

Supporting the east abutment on steel H-piles driven into the hard silty clay to clayey silt till and/or very dense sand and silt till may be considered. However, pre-augering will be required to penetrate the hard/very dense till and provide a sufficient length of pile to achieve lateral fixity.

To achieve an advantageous geotechnical resistance, it is recommended that the piles be driven into the very dense sand and silt till to provide a minimum pile length of 7 m. The following axial geotechnical resistances are recommended for design of H-piles with an assumed tip level of Elev. 273.0, driven to practical refusal in the very dense till:

Table 8.5 – Axial Geotechnical Resistance of Steel H-Piles

	HP 310x110	HP 360x132
Factored Geotechnical Resistance at ULS (kN)	1,400	1,700
Factored Geotechnical Resistance at SLS (kN)	1,200	1,500

Provided the centre to centre pile spacing is no less than three times the pile diameter, a group efficiency of 1.0 may be assumed for pile groups driven into the very dense till.

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

Pile installation should be in accordance with OPSS.PROV 903.

Pile driving must be controlled by the Hiley Formula and an ultimate pile resistance should be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The appropriate pile driving note is "PILES TO BE DRIVEN IN ACCORDANCE WITH STANDARD SS103-11 USING AN ULTIMATE GEOTECHNICAL RESISTANCE OF "R" kN PER PILE". "R" must have a value of two times the design load at +ULS calculated by the structural engineer.

Pile tip protection is recommended for driven H-piles to prevent pile damage when driving through hard clay till and very dense sand and silt till containing cobbles or boulders. The tips of all driven H-piles must be fitted with pile tip protection from an approved manufacturer such as Titus Steel (Standard H-point) or an approved equivalent.

The use of driven steel H-piles is not recommended to support the piers and west abutment based on the following considerations:

- The depth to bedrock is relatively shallow, and consequently the driven pile length may be inadequate for design.
- Driving of H-piles to bedrock may be problematic due to the presence of cobbles, boulders and rock fragments in the sand and gravel deposits and silty sand till layer overlying the bedrock.
- Depending upon the relative frequency and size of boulders, the piles may encounter refusal above the bedrock surface or be damaged during driving.
- The bedrock is highly weathered and contains voids. Piles may encounter refusal on bedding layers or rock slabs overlying voids or highly weathered zones unsuitable for support of the pile tip.

In view of these concerns, the driven pile option has not been developed for the piers and west abutment.

8.5 Socketed H-Pile Foundations

Consideration may be given to supporting the piers and west abutment on steel H-piles socketed into the bedrock. The upper 3.5 to 5.5 m of the bedrock at the site is highly weathered, of poor quality, and contains gypsum layers and solution cavities. Installation of the piles would therefore involve augering to the bedrock surface, augering and/or coring as required to form a socket in the less weathered bedrock at depth, inserting the pile, and grouting the annular space in the socket with concrete.

Socketing operations may be difficult and significantly impacted by the presence of boulders and rock fragments in the sand, gravel and till deposits, as well as by the highly weathered condition of the upper part of the bedrock. A temporary liner will be required to support the auger hole in the cohesionless deposits. In view of the potential installation difficulties and the probable number of piles required, socketed piles are not expected to be preferred over a lesser quantity of high capacity socketed caissons.

A factored geotechnical resistance at ULS of 2,000 kN per pile is recommended for steel HP 310x110 piles socketed at least 1.5 m into the less weathered bedrock. The SLS resistance will not govern design of piles founded on bedrock.

The socket diameter should be approximately 200 mm larger than the largest dimension (corner to corner) of the pile section, and extended to the following tip elevations:

Table 8.6 – Recommended Pile Socket Base Elevations

Foundation Unit	**Reference Boreholes**	**Pile Socket Base Elevation**	
		North End	**South End**
West Abutment	WA-01 to WA-03	259.5	261.0
Pier 1	P1-01 to P1-04	260.0	259.5
Pier 2	P2-01 to P2-04	260.5	260.5
Pier 3	P3-01 to P3-04	263.5	263.5

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

8.6 Drilled-in Pipe Piles

The piers and west abutment may be supported on drilled-in steel pipe piles socketed into bedrock and filled with concrete. This option involves installing the piles using a rotational method such as the Symmetrix concentric drilling system and requires a rock cutting shoe at the tip of the pipe pile.

To penetrate the upper highly weathered zone of the bedrock, fix the pile tip in place, and achieve a resistance value practical for design, it is recommended that the piles be advanced a minimum 1.5 m into the less weathered bedrock at depth. The recommended pile base elevations are the same as those presented in Table 8.6.

The capacity of the drilled-in pipe pile will be dictated by the structural resistance of the composite pile section, and will not be governed by the geotechnical resistance of the bedrock. The factored axial resistances at ULS recommended for several pipe pile sections (concrete-filled) drilled into bedrock are presented in Table 8.7. The SLS resistance will not govern design.

Table 8.7 – Recommended Axial Resistances for Drilled-in Pipe Pile Design

Pipe Pile Section		Factored Axial Resistance at ULS (kN)
Outer Diameter (mm)	Wall Thickness (mm)	
324	9.5	1,650
457	12.7	3,300
610	12.7	5,800
762	15.6	8,350

The axial resistance values assume a steel yield strength of 245 MPa and a concrete compressive strength of 30 MPa. The computed capacity includes a reduction factor of 75% as per Clause 6.11.4.4 of the CHBDC 2014, and assumes a long-term corrosion loss of 2 mm of the pipe sidewall thickness. The structural resistance of the pile must be reviewed by the structural designer. The resistance may need to be further reduced to account for section loss resulting from potential corrosion of the piles.

The depth of the socket may need to be greater than 1.5 m to address the lateral resistance requirement, base fixity requirement and shear and moment demand for each pile.

Installation of pipe piles must follow OPSS.PROV 903 specifications.

The Contractor’s drilling method must be capable of dislodging, removing or penetrating obstructions such as cobbles, boulders or rock fragments in the overburden soils. Care must be exercised while drilling into the bedrock; the drilling methodology must be capable of advancing the pile without disturbing or fracturing the bedrock at the base of the pile. Blasting to facilitate rock removal is not permitted.

Since the rock cutting shoe at the tip of a pipe pile will be slightly larger in diameter than the outside diameter of a pipe pile, there will be a small gap between the rock socket wall and the pipe pile. It is recommended that the annular space between the pipe pile and socket wall be grouted to the bedrock surface to achieve fixity.

During and subsequent to installation, the pipe pile will be partially filled with water and it may not be practical to dewater the pipe prior to concreting. Tremie concreting will be required for concreting these pipe piles.

A NSSP addressing the above issues is included in Appendix F.

8.7 Micropiles

Micropiles socketed into the bedrock could be considered for the piers and west abutment, as installation of smaller diameter elements may be less impacted by the bouldery material and bedrock conditions than larger diameter caissons. Experience with the use of micropiles to support large bridge structures carrying major highways is limited in local practice however, and MTO should be consulted to determine their willingness to consider the use of micropiles.

It is recommended that the micropiles be extended into the less weathered bedrock underlying the upper 3.5 to 5.5 m of highly weathered bedrock, to obtain axial compressive support along the rock socket.

The grout-to-rout bond stress recommended for design of the micropiles within the less weathered bedrock is 350 kPa at factored ULS. For assumed micropile designs with socket diameters of 225 and 305 mm and grouted bond lengths of 5 and 10 m in the less weathered bedrock, the recommended micropile resistances in compression are as follows:

Table 8.8 – Geotechnical Resistance of Micropiles

Socket Diameter (mm)	Grouted Bond Length (m)	Factored Geotechnical Resistance at ULS (kN)
225	5 10	1,240 2,475
305	5 10	1,675 3,350

The interpreted top of the less weathered bedrock to be used in determining the micropile tip elevation is summarized in Table 8.3. The grouted bond length and diameter may be varied as appropriate to achieve different design capacities than shown in the table, and to accommodate equipment selection by the contractor.

The factored SLS resistance of the micropiles in the bedrock socket is expected to exceed the factored ULS resistance as the movement required to develop the bond stress is expected to be less than 10 mm. Axial compression of the unbonded length of the micropile section should be computed by the structural designer.

The allowable tensile capacity may be taken as 75% of the compressive capacity.

The lateral capacity of the micropile will be limited by the slenderness of the installation and the variable nature of the floodplain deposits. Sufficient micropiles should be installed with a batter to resist lateral loads.

The micropiles must be provided with corrosion protection.

Micropile installation will require advancing a hole through cohesionless sand and gravel deposits with cobbles and boulders, till materials containing cobbles and boulders, and highly weathered bedrock containing voids, clay seams and highly fractured zones, prior to encountering less weathered bedrock. The bedrock consists of shale with hard dolostone and limestone layers. The Contractor must be prepared to penetrate these materials to advance the micropile into bedrock and form the rock socket.

It is anticipated that a casing sealed into the less weathered bedrock will be required to support the excavation sidewalls in the cohesionless overburden soils and upper highly weathered portion of the bedrock, prevent materials from falling into the socket, and minimize loss of grout above the bonded zone.

The geotechnical load capacity of a micropile is highly sensitive to the processes used during pile construction, including techniques used for drilling the pile shafts, flushing the drill cuttings and grouting the pile. Micropile load tests prior to and during construction are essential for verification of the assumed grout-to-shale bond stresses. It is recommended that at least two sacrificial piles be installed on each side of the river prior to construction to develop appropriate installation methods and confirm the bond stress/micropile design. Preconstruction and production load testing should be considered an extension of the design.

The Contractor must submit the proposed installation method for review prior to construction. All micropile testing and installation should be witnessed and inspected by qualified geotechnical personnel.

8.8 Recommended Foundation Type

From a geotechnical perspective, the preferred foundation option to support the piers and west abutment of the replacement bridge comprises caissons socketed into bedrock. The use of caissons may expedite construction and minimize site impacts related to extensive cofferdam construction, dewatering, and potential environmental concerns related to construction of spread footings or pile caps in the river.

We understand that the use of micropiles may be advantageous to enable foundation construction to proceed under the existing bridge, to accommodate in-water work timing restrictions. Micropiles are considered feasible from a geotechnical viewpoint, however MTO approval will be required.

The preferred foundation system at the east abutment consists of spread footings founded on hard/very dense till forming the east valley slope. However, the use of augered caissons founded in the till or on bedrock could be considered depending upon the position of the new abutment relative to the existing bridge foundations and the face of the river valley slope.

8.9 Lateral Resistance of Piles and Caissons

The geotechnical lateral resistance of a pile/caisson in the cohesionless riverbed deposits and sand/silt till soil may be calculated using a coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h\, z / D \qquad (\text{kN/m}^3)$$

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

where z = depth of embedment along pile (m)

D = pile width or diameter (m)

n_h = coefficient related to soil density (kN/m^3)

γ' = effective unit weight (kN/m^3)

K_p = coefficient of passive lateral earth pressure

The lateral resistance developed in the hard silty clay to clayey silt till at the east abutment may be calculated using the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) estimated as follows:

$$k_s = 67\ S_u / D \qquad (\text{kN/m}^3)$$

$$p_{ult} = 9\ S_u \qquad (\text{kPa})$$

where: D = pile width/caisson diameter in metres

S_u = undrained shear strength (kPa)

The above equations and recommended parameters in Table 8.9 below may be used to analyse the interaction between a pile/caisson and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance.

Table 8.9 – Soil Parameters for Lateral Pile Design

Soil Type	**γ' (kN/m^3) ***	**S_u (kPa)**	**n_h (kN/m^3)**	**K_p**
Hard silty clay to clayey silt till	21	200	-	3.0
Variable sand and gravel, gravelly sand and silty sand river deposits	10	-	3,000	3.2
Very dense silty sand to sandy silt till	11		10,000	3.7

*Buoyant unit weight below the water table.

The spring constant, K_s, for analysis may be obtained by the expression, $K_s = k_s L D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the pile/caisson width (m) and L is the length (m) of the pile/caisson segment or element used in the analysis. The ultimate lateral resistance, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult} L D$. This represents the ultimate load at which geotechnical failure of the pile/caisson occurs and will not support any additional load at greater displacement.

The modulus of subgrade reaction may have to be reduced based on the pile/caisson spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Figure C6.11.3 in the Commentary to the CHBDC 2014.

The lateral resistance that can be mobilized in front of a caisson socket in bedrock may be computed using the coefficient of horizontal subgrade reaction k_s and ultimate lateral resistance p_{ult} values provided below.

k_s = 35,000 kN/m^3 in the highly weathered bedrock; and
= 75,000 kN/m^3 in the less weathered bedrock.

p_{ult} = 650 kPa in the highly weathered bedrock; and
= 1,500 kPa in the less weathered bedrock.

9.0 FROST COVER

The depth of frost penetration at this site is 1.4 m, as per OPSD 3090.101. The base of footings or pile caps must be provided with a minimum of 1.4 m of earth cover as protection against frost action.

10.0 SEISMIC CONSIDERATIONS

In accordance with the CHBDC, the selection of the seismic site class is based on the soil conditions encountered in the upper 30 m of the ground profile. The stratigraphy at this site generally consists of surficial fill, sand and gravel river deposits, and hard/dense to very dense tills underlain by shale/dolostone bedrock at depths of less than 20 m. As per Table 4.1, Clause 4.4.3.2 of the CHBDC (2014), the site may be classified as Seismic Site Class C (very dense soil and soft rock).

Based on the National Building Code of Canada (NBCC 2015), the peak horizontal ground acceleration (PGA), corresponding to a design earthquake having a 2 percent probability of being exceeded in 50 years (i.e. 2,475 year return period) is 0.082 g at the site.

Based on review of the SPT data, seismically-induced liquefaction of foundation soils is not anticipated under the design earthquake.

11.0 ABUTMENT BACKFILL AND LATERAL EARTH PRESSURES

Backfill to the abutments should consist of free-draining granular material conforming to OPS Granular A or B Type II specifications. The granular material should be placed to the extents shown in OPSD 3101.150. Compaction should be carried out in accordance with OPSS.PROV 206 and OPSS.PROV 501.

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

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

Where:
p = horizontal earth pressure on the wall at depth h (kPa)
K = earth pressure coefficient (see table below)
γ = unit weight of retained soil (see table below)
h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

The earth pressure coefficients are dependent on the material used as backfill. Recommended unfactored values are shown in Table 11.1. The at-rest coefficients should be employed for restrained walls. Active pressures should be used for any wingwalls or unrestrained walls.

Table 11.1 – Lateral Earth Pressure Coefficients

Loading Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or Granular B Type II $\phi = 35°, \gamma = 22.8\ kN/m^3$		OPSS Granular B Type I $\phi = 32°, \gamma = 21.2\ kN/m^3$	
	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.39*	0.31	0.47*
At-rest (Restrained Wall)	0.43	-	0.47	-
Passive	3.7	-	3.3	-

* For wing walls.

The parameters in the table correspond to full mobilization of active and passive earth pressures and require certain relative movements between the wall and adjacent soil to produce these conditions. The values to be used in design can be assessed from Figure C6.16 of the Commentary to the CHBDC.

The use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) is generally preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. Granular B, Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves towards the soil mass.

In accordance with Clause 6.12.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 1.7 m for Granular B Type I or 2.0 m for Granular A or Granular B Type II.

The impact of seismic-induced forces on the abutments should be assessed in accordance with Section C4.6.5 of the Commentary to the CHBDC.

The design of the abutment walls must incorporate measures such as weep holes and/or subdrains to permit drainage of the backfill and avoid the potential build-up of hydrostatic pressures behind the walls.

12.0 EMBANKMENT SLOPES

The west bridge approach is located on an approximate 5.5 m high fill embankment constructed within the wide river floodplain, and the east approach is located within an approximate 8 to 9 m deep earth cut excavated into the east valley wall. Widening of the highway on the new bridge approaches will require widening of both the west approach embankment and the east cut section.

Embankment widening should be carried out in accordance with OPSS.PROV 206. Materials used to construct the embankment widening should comprise granular materials or Select Subgrade Material (SSM) in compliance with OPSS.PROV 1010. Where new embankment fill is placed against the existing embankment slopes of the west approach, the existing fill slope must be benched in accordance with OPSD 208.010.

Assessment of the high fill embankment slopes at the west approach is presented in a separate Foundation Investigation and Design Report. The embankment slopes are expected to be stable with side slopes inclined no steeper than 2H:1V. Mid-height berms comprising 2 m wide benches should be incorporated along the length of embankments with heights exceeding 8 m. Settlement of the embankments due to compression of the foundation subgrade is generally expected to be less than 25 mm, provided all topsoil, peat and organic materials are removed from the embankment footprint.

Assessment of the cut slopes at the east approach is also presented in a separate Foundation Report. In general, permanent roadway cuts along the east approach are expected to be stable with side slopes constructed no steeper than 2H:1V in the native soils. Flatter slopes may be required where loose soils or groundwater seepage (from perched zones or the regional groundwater table) is encountered. Earth cut slopes greater than 6 m high should be provided with a 2 m wide mid-height berm.

Embankment slopes must be provided with erosion protection in accordance with OPSS.PROV 804. Typically, rock protection should be provided over all surfaces with which river flow is likely to be in contact. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion. Surface water should be directed away from the embankment slopes and conveyed down the slope in appropriately designed drainage channels or storm sewers.

13.0 EROSION AND SCOUR PROTECTION

Footings and pile caps at the piers and west abutment must be protected from scour by creek flow, considering both flooding conditions and potential morphological changes in stream channel alignment. Pier construction must not alter creek flow directions in a way that directs flow towards the ravine valley slopes and increases erosion rates.

Erosion and scour protection measures should be designed by a qualified and experienced river and/or hydraulic engineering specialist.

Any water flows (drainage and/or storm water) from sub drains, the bridge structure or roadway ditches must be appropriately conducted to the base of the slope to avoid development of erosion gulleys on the slope surface. Care should be taken to protect the existing vegetation on the adjacent slopes.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS.PROV 804.

14.0 EXCAVATION AND GROUNDWATER CONTROL

All excavation must be carried out in accordance with OPSS.PROV 902 and the Occupational Health and Safety Act (OHSA). For the purposes of assessing excavation slope requirements in compliance with the OHSA, the following soil types should be assumed:

- The fill materials are classified as Type 3 soils above the water level and Type 4 below.
- The cohesionless sand and gravel, gravelly sand and silty sand deposits within the floodplain are classified as Type 3 soils above the water level and Type 4 below.
- The dense to very dense till layer underlying the floodplain deposits is classified as Type 2 soil; and
- The very dense sand, hard silty clay to clayey silt till, and very dense sand and silt till at the east abutment are classified as Type 1 soils.

Use of a hydraulic excavator should be suitable for excavation in the overburden soils. The selection of the method of excavation is the responsibility of the contractor and must be based on their equipment, experience and interpretation of the site conditions. Provision must be made for the handling of potential obstructions in the existing fill materials, numerous cobbles, boulders or other obstructions in the floodplain deposits, and cobbles and boulders in the till. Laboured excavation should be anticipated in the hard/dense to very dense native soils.

It is expected that excavation of the weathered bedrock will require heavy excavation equipment equipped with a rock bucket and rippers, supplemented by pneumatic rock breakers. Intensive use of pneumatic/hydraulic breakers or other methods may be required in more sound bedrock at depth or to penetrate hard limestone layers. Progressively more difficult conditions should be anticipated with increasing depth of excavation.

Roadway protection should be provided at the approaches in accordance with OPSS.PROV 539 as amended by SP105S09, and designed for Performance Level 2. Based on available subsurface information, a shoring system consisting of sheet piling or steel H-piles with timber lagging may be considered. Cobbles and/or boulders, as well as very dense conditions, may be encountered during installation of the protection systems, and the contractor should be prepared to predrill to loosen the soils and push obstructions aside, or to otherwise remove or penetrate the obstructions.

Excavation for footing or pile cap construction within the river flood plain will require cofferdam installation and dewatering. Dewatering equipment may need to handle significant flow volumes in view of the permeable nature of the weathered bedrock and overlying granular deposits. High volume sumps installed within the excavation in conjunction with interlocking steel sheet piling cutoff around the foundation excavation may provide a suitable system. The groundwater control measures must be implemented prior to commencing excavation below the river water level.

Based on the results of the boreholes drilled within the river valley, installation of sheet pile cofferdams may be problematic due to the frequency of cobbles and/or boulders, as well as the locally very dense conditions. Predrilling may be required to loosen the soils and push obstructions aside from the pile alignment. The sheet piles should be driven to bedrock if possible. Suggested wording for an NSSP to alert the contractor to these conditions is provided in Appendix F. Subject to environmental restrictions and conservation authority approval, the cofferdam may be left in place below the riverbed.

The design of the dewatering system is the responsibility of the Contractor, and the Contract Documents must alert them to this responsibility. The design must be in accordance with OPSS.PROV 902 as amended by NSSP FOUN0003, and OPSS.PROV 517 as amended by SP517F01. As water levels in the river and adjacent lands would not be lowered by dewatering within a cofferdam, a preconstruction survey is not required and Designer Fill-In ****** in SP

FOUN0003 should be “N/A”. Considering the conditions on site, a design Engineer and design-checking Engineer with a minimum of 5 years of experience in designing systems of similar nature and scope to the required work is required, and thus Designer Fill-In ***** in SP517F01 should be “Yes”.

It is anticipated that registering with the Environmental Activity and Sector Registry (EASR) or obtaining an MECP Permit to Take Water (PTTW) will be required and should be coordinated prior to construction.

The selection and design of the temporary protection systems, excavation operations, and dewatering procedures are the responsibility of the contractor. The protection systems should be designed by a licensed Professional Engineer experienced in design of shoring with consideration of adjacent traffic loads, construction operations, and any sloping retained surfaces. A dewatering specialist should be consulted to provide input on the required dewatering system.

15.0 SOIL CORROSION POTENTIAL

The low resistivity value and elevated chloride concentration measured in a sample of fill from the west approach (Borehole WA-04) indicate a corrosive environment for steel, cast iron, and other metals (MTO Gravity Pipe Design Guidelines, 2014). An elevated chloride concentration was also measured in the native sample from Borehole P1-02. In view of these results and the ongoing application of de-icing salt to the highway, protective measures to resist corrosion should be provided.

The measured sulphate concentrations indicate that buried concrete structures will not be subject to sulphate attack in the overburden soils (CSA A23.1-14). Given the occurrence of gypsum inclusions in the bedrock, sulphate resistant concrete should be used for all foundation elements in contact with the bedrock. The potential for corrosion of steel piles must also be taken into consideration.

16.0 CONSTRUCTION CONCERNS

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

- Staging of the bridge replacements must be carried out in a manner that minimizes the potential for disturbance of functioning bridge foundations adjacent to the work area. The GA drawing for the existing bridge indicates that the piers and west abutment are supported on bedrock and the east abutment is supported on footings on native soil above the east valley slope. Foundation installation activities must consider the potential for disturbance to the subgrade on which these foundations are constructed.

- Construction of caissons or micropiles to support the new bridges will extend below the bedrock surface and may be in close proximity to the existing foundations. A monitoring program should be implemented for the duration of new foundation construction to identify any movement of the existing structure. The program should include establishment of appropriate monitoring points on the existing structure, acquirement of baseline readings in advance of construction, specification of tolerable levels of movement by the structural designer, and development of remedial actions if movements exceed tolerable levels.

- The footings supporting the existing bridge may interfere with new footing construction or caisson/pile installation. It is recommended that the new foundation units be positioned to avoid the existing foundations, the existing foundations be removed prior to installation of new foundations, and/or the contract specifically alert the contractor (such as through a Notice to Contractor) of the need to penetrate the existing foundations during construction of the new foundations.

- The river valley deposits are locally very dense and contain cobbles and boulders. In addition, the soils underlying the east valley slope are hard/very dense. These conditions may have a significant impact on excavation activities, pile driving, socket or caisson augering, and sheet pile installation. The feasibility of the various construction activities and measures to mitigate the impact will need to be considered during selection of construction procedures.

- At the piers and west abutment within the river flood plain, excavation for foundation construction will require dewatering and excavation support systems such as sheet pile cofferdams. In view of the highly weathered nature of the bedrock, significant inflow of water may be experienced even with sheet piles driven to the bedrock surface.

- It is expected that permanent steel liners will be required during caisson construction to support the excavation sidewalls in the cohesionless valley base deposits.

- The highly weathered bedrock contains voids, clay seams and highly fracture zones. Augering and socketing operations may be difficult and significantly impacted by these conditions. The Contractor must be prepared to penetrate these materials to advance the caisson into bedrock and form the rock socket. It is anticipated that a caisson liner will be required to support the excavation sidewalls in the cohesionless overburden soils and upper highly weathered portion of the bedrock, and prevent materials from falling into the excavation.

- High volumes of seepage should be anticipated into excavations extended into bedrock, and measures such as heavy duty pumping to maintain a dry excavation and enable concrete placement in a dewatered condition may not be practical. It is anticipated that placement of concrete using tremie methods will be required.
- An artesian groundwater condition was identified locally in the bedrock at the east pier. Concrete placement for caisson/socket construction at this location must include measures (such as maintaining a head of water in the liner) to counter potential impacts of the artesian pressures on the integrity of tremied concrete.
- Given the occurrence of gypsum inclusions in the bedrock, sulphate resistant concrete should be used for all foundation elements where applicable.

17.0 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Mr. Karel Furbacher, P.Eng. and Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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Appendix A

Site Photographs

Grand River Bridge Replacement
Site Photographs

Photograph 1 – South side of bridge looking west from east abutment

Photograph 2 – East approach looking east from south end of east abutment

Photograph 3 – North side of bridge looking west from east bank of river

Photograph 4 – West pier and abutment, north side of bridge

Photograph 5 – North side of bridge looking east from west river bank

Photograph 6 – South side of bridge looking east from west river bank

Photograph 7 – Access mats on north side of bridge, from west approach

Photograph 8 – Access mats for Pier 2 drilling, facing north

THURBER

Appendix B

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction
1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT value
5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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

▼ Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

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

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

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

SYMBOLS

▨	CLAYSTONE
▤	SILTSTONE
░	SANDSTONE
■	COAL
▩	Bedrock (general)

DISCONTINUITY SPACING

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

TERMS

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

STRENGTH CLASSIFICATION

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

Ministry of Transportation
Ontario

THURBER

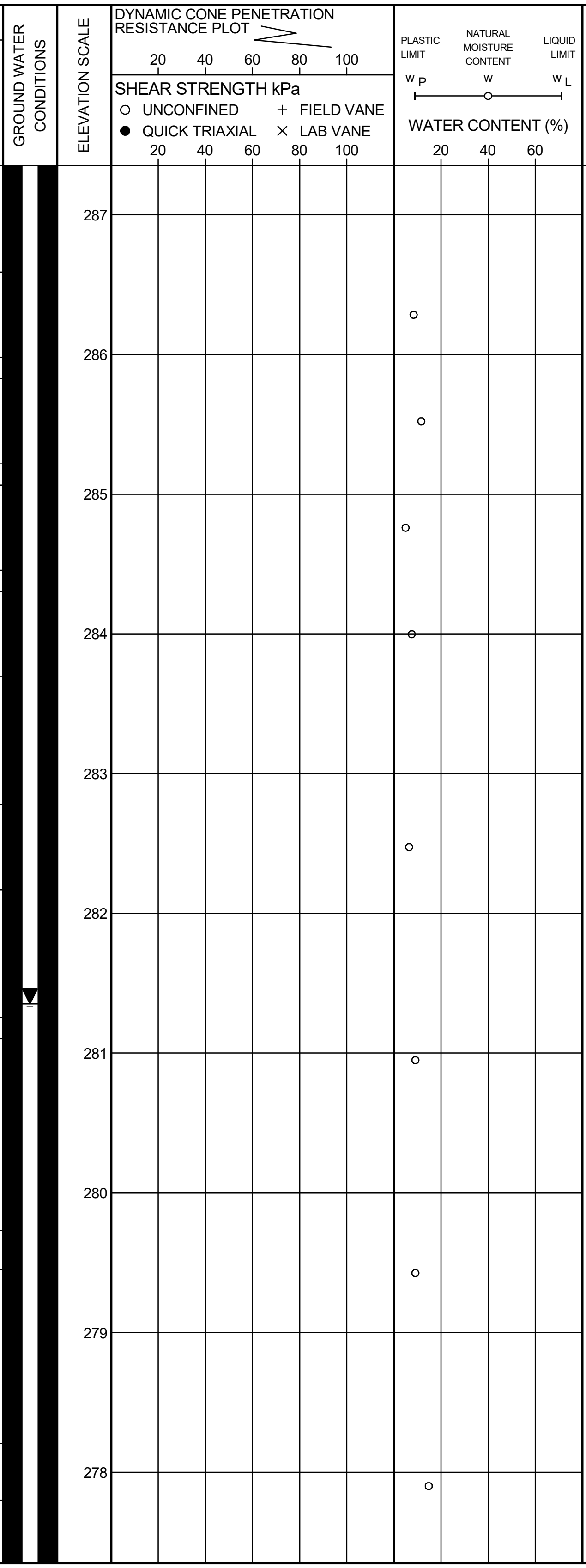
RECORD OF BOREHOLE No EA-01

1 OF 3

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 927.5 E 233 113.8
ORIGINATED BY SJ
DIST
HWY 401
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2018.04.25 - 2018.04.27
LATITUDE 43.398823
LONGITUDE -80.384975
CHECKED BY MRA

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
287.4	GROUND SURFACE							
0.0	**TOPSOIL:** (300mm) (FILL)							
287.0 0.3	**SAND**, some gravel, trace to some silt Dense to Very Dense Brown Moist	1	SS	50				
285.9 1.4	Silty **CLAY**, some sand, trace gravel Hard Brown Moist (TILL)	2	SS	36				
285.1 2.2	**SAND**, trace silt and clay Very Dense Brown Moist	3	SS	66	0	86	14 (SI+CL)	
		4	SS	66				
		5	SS	88				
281.7 5.6	Silty **CLAY** to clayey **SILT**, trace sand to sandy Hard Brown Moist (TILL)	6	SS	100/ 0.150	0	23	53	24
		7	SS	100/ 0.275				
		8	SS	100/ 0.250	0	9	72	19



UNIT WEIGHT γ kN/m³

Continued Next Page

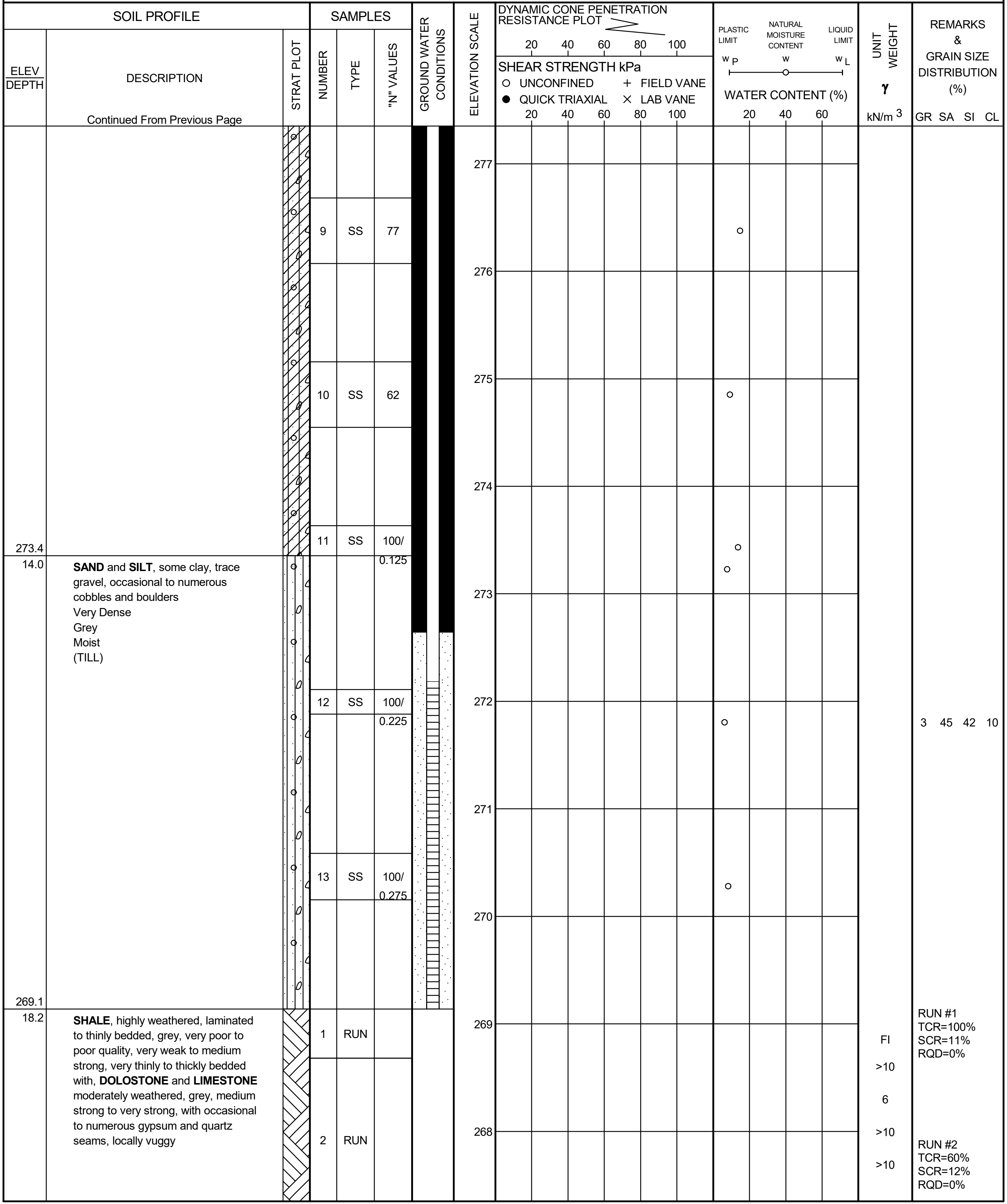
+³, ×³: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No EA-01

2 OF 3 **METRIC**

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 927.5 E 233 113.8
ORIGINATED BY SJ
DIST HWY 401
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2018.04.25 - 2018.04.27
LATITUDE 43.398823
LONGITUDE -80.384975
CHECKED BY MRA



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page					
		9	SS	77		
		10	SS	62		
273.4		11	SS	100/ 0.125		
14.0	**SAND** and **SILT**, some clay, trace gravel, occasional to numerous cobbles and boulders Very Dense Grey Moist (TILL)					
		12	SS	100/ 0.225		3 45 42 10
		13	SS	100/ 0.275		
269.1						
18.2	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy	1	RUN		FI >10 6	RUN #1 TCR=100% SCR=11% RQD=0%
		2	RUN		>10 >10	RUN #2 TCR=60% SCR=12% RQD=0%

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No EA-01

3 OF 3

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 927.5 E 233 113.8
ORIGINATED BY SJ
DIST HWY 401
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2018.04.25 - 2018.04.27
LATITUDE 43.398823
LONGITUDE -80.384975
CHECKED BY MRA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	NUMBER	TYPE	"N" VALUES						
						267				
		3	RUN						>10 >10 >10	RUN #3 TCR=65% SCR=7% RQD=0%
						266				
	Clay seam (50mm) at 20.8m and (25mm) at 21.0m								5 5	
						265				
		4	RUN						9 9 9	RUN #4 TCR=95% SCR=30% RQD=7%
						264			5 6	
		5	RUN			263			7 5 5	RUN #5 TCR=97% SCR=78% RQD=20%
262.6 24.8	END OF BOREHOLE AT 24.8m. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen.									

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.04.27	6.5	280.9
2018.08.09	6.0	281.4
2018.10.22	6.0	281.4

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

$\overset{20}{15 \oplus 5}_{10}$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No EA-02

1 OF 2

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 898.2 E 233 108.9
ORIGINATED BY JM
DIST HWY 401
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY MP
DATUM Geodetic
DATE 2018.07.30 - 2018.07.30
LATITUDE 43.398559
LONGITUDE -80.385031
CHECKED BY MRA

ELEV DEPTH	DESCRIPTION
282.8	GROUND SURFACE
0.0	**SAND**, trace to some silt, trace to some gravel, occasional organics Compact Brown Moist (FILL)
281.8 1.0	Sandy silty **CLAY** to clayey **SILT**, trace gravel, occasional to numerous cobbles and boulders Hard Brown to Grey Moist (TILL)
275.2 7.6	**SAND** and **SILT**, some clay, trace sand, occasional to numerous cobbles and boulders Very Dense Grey Moist (TILL)

SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
1	SS	18					
2	SS	21					
3	SS	85/ 0.225		0	26	46	28
4	SS	85/ 0.275					
5	SS	83/ 0.225					
6	SS	74					
7	SS	94/ 0.275		4	33	44	19
8	SS	77/ 0.275					
9	SS	74/ 0.200	FI				

GROUND WATER CONDITIONS

ELEVATION SCALE: 282, 281, 280, 279, 278, 277, 276, 275, 274, 273

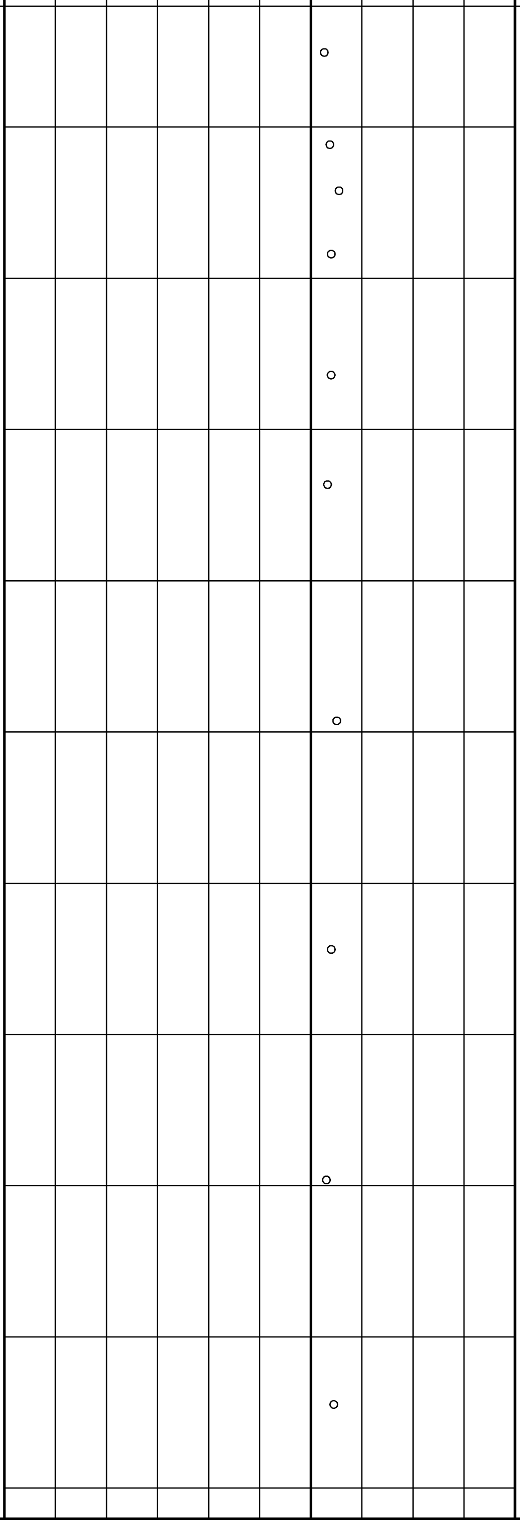
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED
+ FIELD VANE
● QUICK TRIAXIAL
× LAB VANE

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

WATER CONTENT (%)

REMARKS & GRAIN SIZE DISTRIBUTION (%)



Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No EA-02

2 OF 2

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 898.2 E 233 108.9
ORIGINATED BY JM
DIST
HWY 401
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY MP
DATUM Geodetic
DATE 2018.07.30 - 2018.07.30
LATITUDE 43.398559
LONGITUDE -80.385031
CHECKED BY MRA

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%): 20 40 60

SOIL PROFILE		SAMPLES						
ELEV DEPTH	DESCRIPTION Continued From Previous Page	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		1	RUN			272		
270.8		2	RUN			271	>10 >10 3	RUN #2 TCR=57% SCR=25% RQD=10%
12.0	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy							
		3	RUN			270 269	>10 >10 >10	RUN #3 TCR=60% SCR=35% RQD=0%
268.5								
14.3	END OF BOREHOLE AT 24.8m. BOREHOLE CAVED TO 7.3m AND WATER LEVEL AT 3.0m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.							

+ ³, × ³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

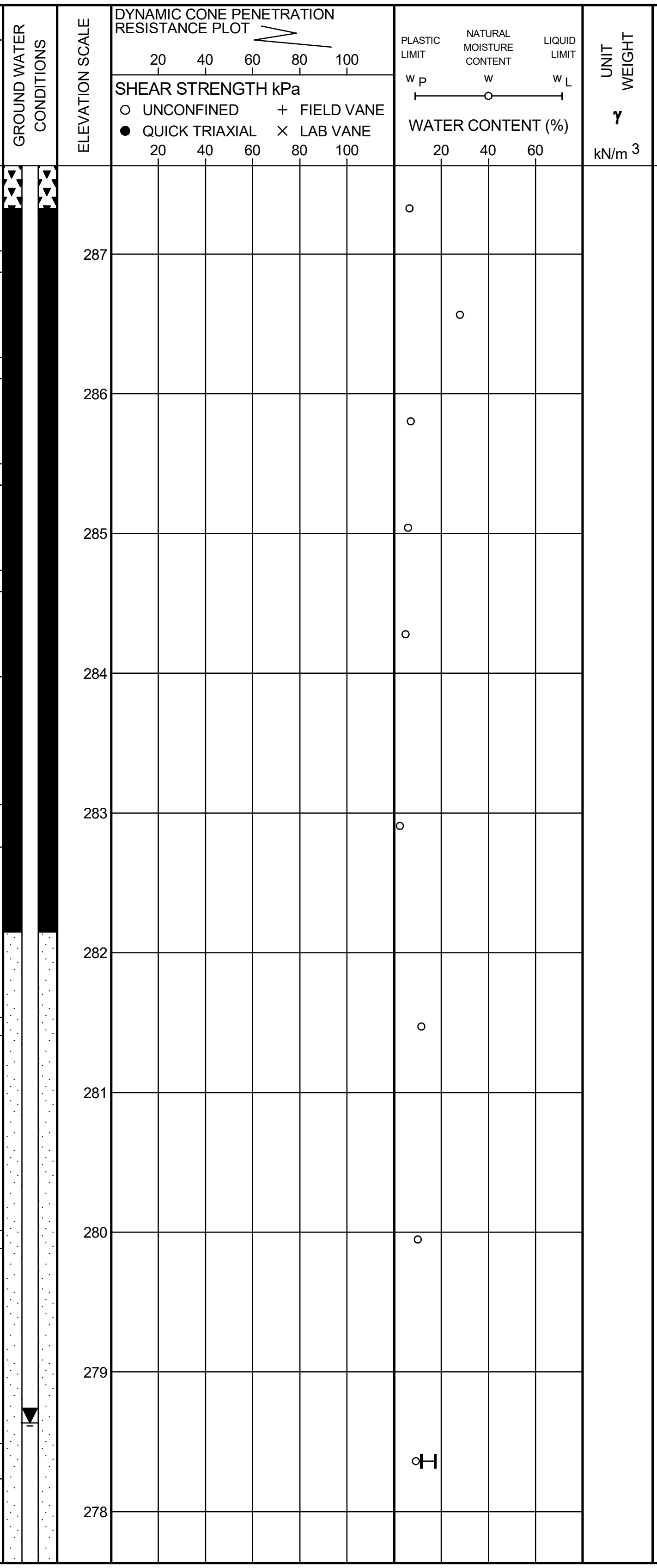
RECORD OF BOREHOLE No EA-03

1 OF 3

METRIC

W.P. 3080-12-02/03	LOCATION MTM NAD 83 Zone 10: N 4 806 887.7 E 233 126.2		ORIGINATED BY SB
DIST HWY 401	BOREHOLE TYPE Hollow Stem Augers/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2017.12.13 - 2017.12.15	LATITUDE 43.398466 LONGITUDE -80.384816	CHECKED BY MRA

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
287.6	GROUND SURFACE							
0.0 287.3	**TOPSOIL:** (300mm) (FILL)	1	SS	15				
0.3	**SAND** and **GRAVEL**, trace silt Compact to Loose Brown Dry to Moist (FILL)							
		2	SS	26				
		3	SS	15	38	47	15 (SI+CL)	
		4	SS	7				
		5	SS	15				
	Very Dense	6	SS	65/ 0.150	46	46	8 (SI+CL)	
282.0 5.6	Sandy clayey **SILT**, trace gravel Hard Grey to Brown Moist (TILL)							
		7	SS	100/ 0.125	3	25	56	16
		8	SS	100/ 0.125				
		9	SS	100/ 0.100	3	31	50	16



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

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No EA-03

2 OF 3

METRIC

W.P. 3080-12-02/03	LOCATION MTM NAD 83 Zone 10: N 4 806 887.7 E 233 126.2		ORIGINATED BY SB
DIST HWY 401	BOREHOLE TYPE Hollow Stem Augers/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2017.12.13 - 2017.12.15	LATITUDE 43.398466 LONGITUDE -80.384816	CHECKED BY MRA

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

ELEV DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page					
277.0 10.7	**SILT** and **SAND**, trace clay, trace gravel, occasional cobbles and boulders Very Dense Brown Moist (TILL)	10	SS	100/ 0.150	277 276	
		11	SS	100/ 0.150	275 274	9 37 45 9
	Layer of **SAND**, trace silt, trace gravel, wet	12	SS	65/ 0.100	273	2 91 7 (SI+CL)
		13	SS	100/ 0.050	272 271	
		14	SS	100/ 0.025	270	
269.8 17.8	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy	1	RUN		269 268	RUN #1 TCR=68% SCR=3% RQD=0% RUN #2 TCR=100% SCR=59% RQD=0%

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No EA-03

3 OF 3

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 887.7 E 233 126.2
ORIGINATED BY SB
DIST
HWY 401
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY MP
DATUM Geodetic
DATE 2017.12.13 - 2017.12.15
LATITUDE 43.398466
LONGITUDE -80.384816
CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
			2	RUN							
			3	RUN			267				RUN #3 TCR=41% SCR=26% RQD=0%
			4	RUN							RUN #4 TCR=59% SCR=0% RQD=0%
			5	RUN			266 265				RUN #5 TCR=96% SCR=63% RQD=14%
263.2			6	RUN			264				RUN #6 TCR=96% SCR=91% RQD=25%
24.5	END OF BOREHOLE AT 24.5m. BOREHOLE OPEN TO 17.1m. Well installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen.										

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.08.09	8.4	279.2
2018.10.22	9.0	278.6

ONTMT4S2 MTO-11373.GPJ 2017TEMPLATE(MTO).GDT 11/21/18

+ ³, × ³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

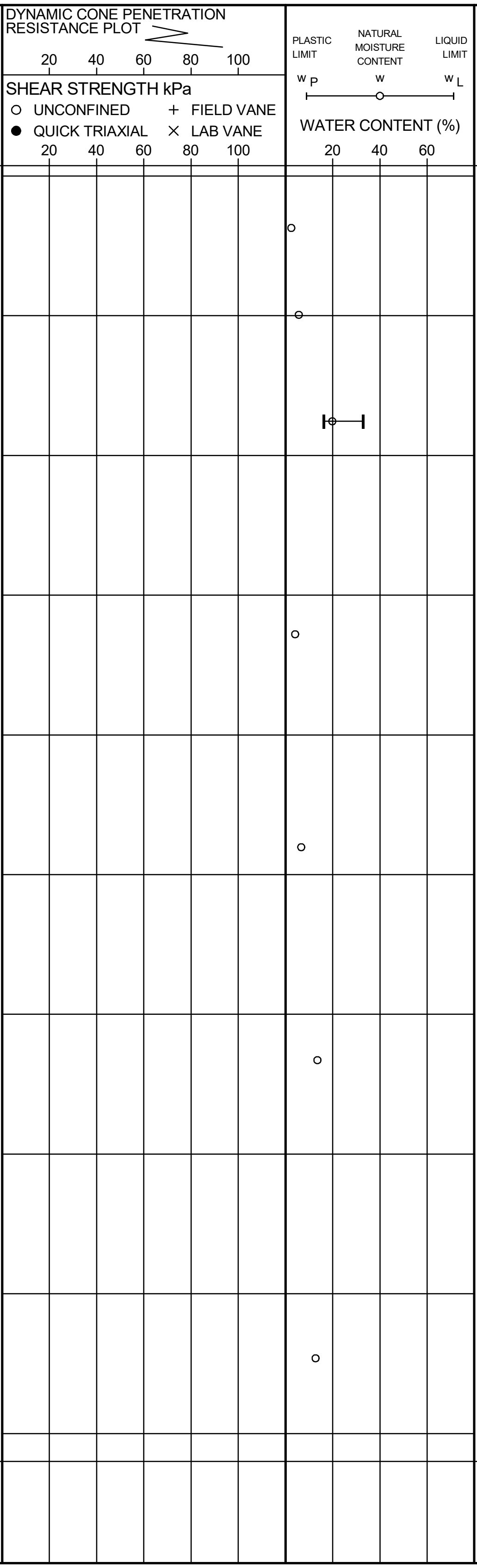
RECORD OF BOREHOLE No EA-04

1 OF 2

METRIC

W.P.	3080-12-02/03
LOCATION	MTM NAD 83 Zone 10: N 4 806 925.4 E 233 120.2
ORIGINATED BY	RP
DIST	
HWY	401
BOREHOLE TYPE	Solid Stem Augers
COMPILED BY	MP
DATUM	Geodetic
DATE	2018.04.30 - 2018.04.30
LATITUDE	43.398805
LONGITUDE	-80.384895
CHECKED BY	MRA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
288.1	GROUND SURFACE					288					
0.0 287.8	**ASPHALT:** (290mm)										
0.3 286.6	**SAND** and **GRAVEL** to gravelly, trace silt Brown Moist (FILL)	1	SS	18		287		26	62	12 (SI+CL)	
1.4 285.9	Silty **CLAY**, some sand, trace gravel Hard Grey Moist (TILL)	2	SS	32		286		6	10	37	47
2.2	**SAND**, trace to some silt and gravel Dense to Very Dense Brown Moist	3	SS	50/ 0.125		285					
		4	SS	47		284					
		5	SS	85		283					
		6	SS	50		282					
280.9	Wet					281					
7.2	Sandy clayey **SILT** Hard Grey Moist (TILL)	7	SS	50/ 0.100		280					
278.8		8	SS	50/ 0.125		279					
9.3	END OF BOREHOLE AT 9.3m. BOREHOLE OPEN AND DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND										



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+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No EA-04

2 OF 2

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 925.4 E 233 120.2
ORIGINATED BY RP
DIST HWY 401
BOREHOLE TYPE Solid Stem Augers
COMPILED BY MP
DATUM Geodetic
DATE 2018.04.30 - 2018.04.30
LATITUDE 43.398805
LONGITUDE -80.384895
CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	CUTTINGS, ASPHALT AT SURFACE.										

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No EA-05

1 OF 1

METRIC

W.P. 3080-12-02/03	LOCATION MTM NAD 83 Zone 10: N 4 806 898.3 E 233 132.1	ORIGINATED BY SB
DIST ____ HWY 401	BOREHOLE TYPE Solid Stem Augers	COMPILED BY MP
DATUM Geodetic	DATE 2017.12.11 - 2017.12.11 LATITUDE 43.398562 LONGITUDE -80.384745	CHECKED BY MRA

SOIL PROFILE: ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
287.9	GROUND SURFACE										
0.0	**ASPHALT:** (200mm)										
0.2	**SAND** and **GRAVEL**, trace to some silt Very Dense Brown Moist (FILL)		1	SS	100/ 0.275						
			2	SS	100/ 0.275		287				
286.7 1.2	**SAND**, some gravel to gravelly, trace to some silt Very Dense Brown Moist		3	SS	95		286	29	57	14 (SI+CL)	
			4	SS	100/ 0.125		285				
			5	SS	100/ 0.250		284				
			6	SS	100/ 0.075		283	18	56	26 (SI+CL)	
282.6 5.4	END OF BOREHOLE AT 5.4m. BOREHOLE OPEN AND DRY. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS, ASPHALT AT SURFACE.		7	SS	100/ 0.025						

DYNAMIC CONE PENETRATION RESISTANCE PLOT

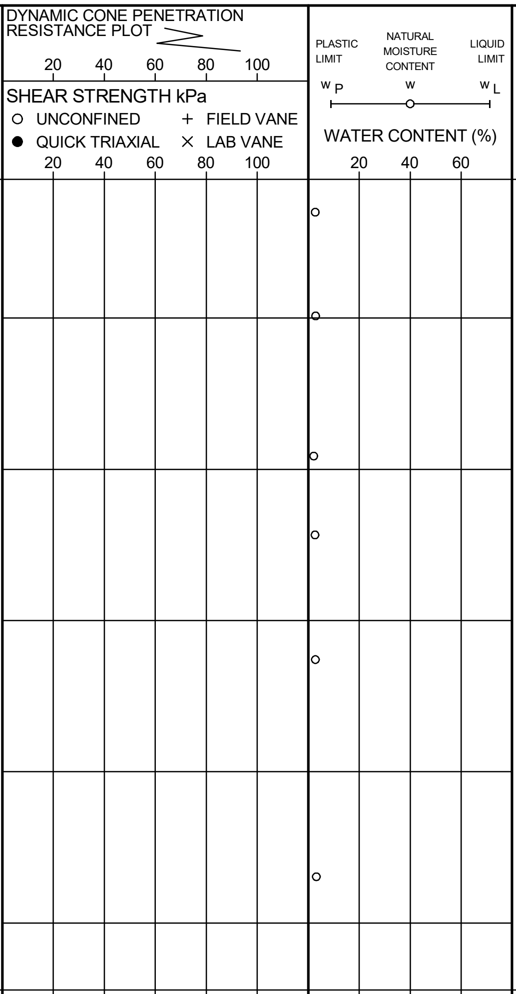
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

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

WATER CONTENT (%)

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)



+ ³, × ³: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No P1-01

1 OF 2

METRIC

W.P.	3080-12-02/03	LOCATION	MTM NAD 83 Zone 10: N 4 806 818.7 E 232 944.1	ORIGINATED BY	AF
DIST	HWY 401	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2018.04.04 - 2018.04.04 LATITUDE 43.397828 LONGITUDE -80.387055	CHECKED BY	MRA

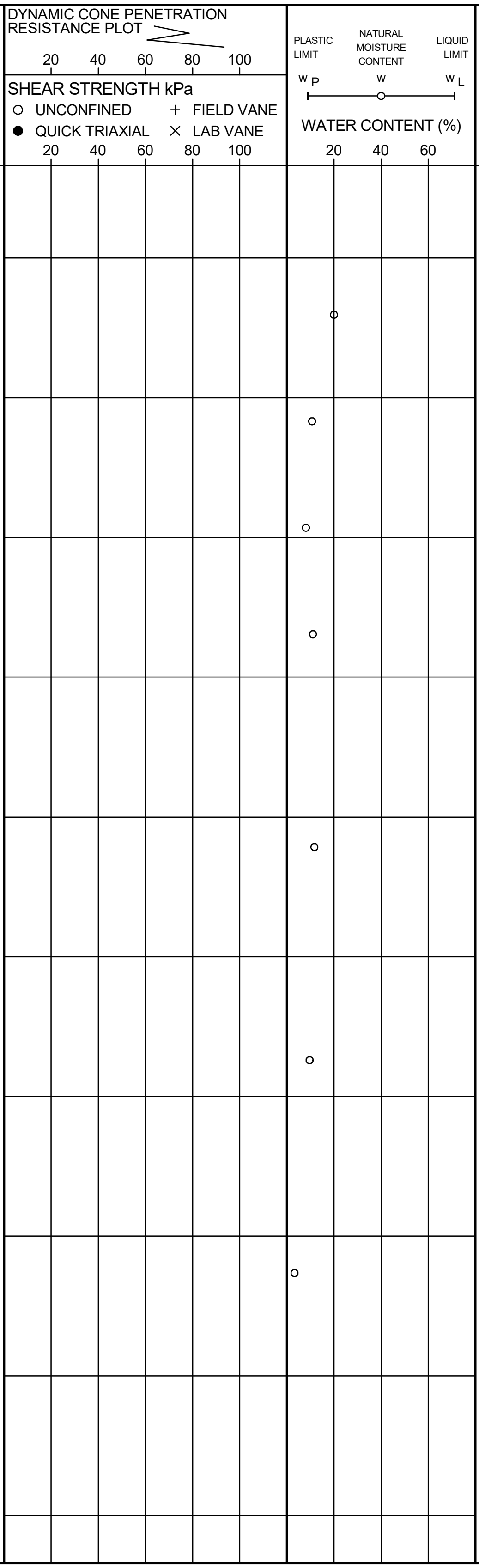
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
273.7	GROUND SURFACE
0.0 273.4	**ORGANICS/TOPSOIL:** (300mm)
0.3 269.5	**SAND** and **GRAVEL**, trace silt, occasional to numerous cobbles and boulders Compact Brown Moist
4.2 266.0	Silty **SAND**, trace clay and gravel Dense to Very Dense Brown Moist (TILL)
7.6	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	22					
2	SS	38		65	24	11 (SI+CL)	
3	SS	14					
4	SS	21		49	50	1 (SI+CL)	
5	SS	47					
6	SS	100/ 0.100					
7	SS	100/ 0.050					
1	RUN		FI >10 >10 >10	RUN #1 TCR=58% SCR=15% RQD=0%			
			>10	RUN #2 TCR=52% SCR=13% RQD=0%			

GROUND WATER CONDITIONS



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+ 3, × 3: Numbers refer to Sensitivity

20
15 5
10
(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No P1-01

2 OF 2

METRIC

W.P.	3080-12-02/03	LOCATION	MTM NAD 83 Zone 10: N 4 806 818.7 E 232 944.1	ORIGINATED BY	AF
DIST	HWY 401	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2018.04.04 - 2018.04.04 LATITUDE 43.397828 LONGITUDE -80.387055	CHECKED BY	MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
		▨	2	RUN			263			>10 >10	
262.6 11.0	END OF BOREHOLE AT 11.0m. Well installation consists of 50mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.										

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.04.05	0.8	272.9
2018.04.09	1.0	272.7
2018.04.11	1.1	272.6
2018.10.22	1.3	272.4

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

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No P1-02

1 OF 2

METRIC

W.P. 3080-12-02/03

LOCATION MTM NAD 83 Zone 10: N 4 806 806.0 E 232 947.5

ORIGINATED BY AF

DIST HWY 401

BOREHOLE TYPE Hollow Stem Augers/NQ Coring

COMPILED BY AN

DATUM Geodetic

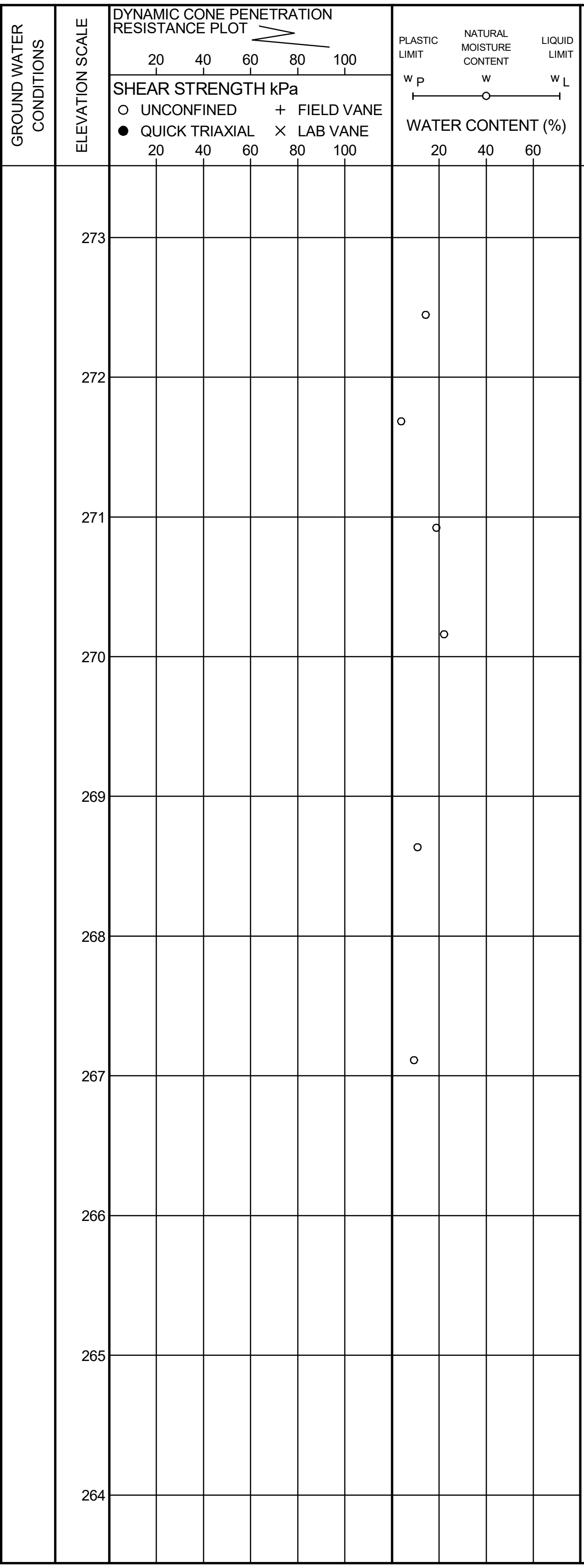
DATE 2018.04.04 - 2018.04.05

LATITUDE 43.397714

LONGITUDE -80.387012

CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
273.5	GROUND SURFACE					
0.0 273.2	**TOPSOIL:** (275mm)					
0.3	**SAND** and **GRAVEL**, trace to some silt, occasional to numerous cobbles and boulders, occasional organics Compact to Loose Brown Moist	1	SS	19		
		2	SS	8		
		3	SS	7		
		4	SS	5		
		5	SS	13		
		6	SS	22		37 54 9 (SI+CL)
266.2 7.3	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy	1	RUN		FI 7 >10 >10 >10	RUN #1 TCR=82% SCR=18% RQD=0%
		2	RUN		>10 >10	RUN #2 TCR=37% SCR=4% RQD=0%



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+ 3, × 3: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No P1-02

2 OF 2

METRIC

W.P.	3080-12-02/03	LOCATION	MTM NAD 83 Zone 10: N 4 806 806.0 E 232 947.5	ORIGINATED BY	AF
DIST	HWY 401	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2018.04.04 - 2018.04.05 LATITUDE 43.397714 LONGITUDE -80.387012	CHECKED BY	MRA

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	slightly weathered to fresh, fair to good quality										
			3	RUN			263 262			>10 5 >10	RUN #3 TCR=55% SCR=25% RQD=7%
			4	RUN			261			>10 2 1 0 0	RUN #4 TCR=92% SCR=73% RQD=70%
			5	RUN			260 259			0 0 2 1 0	RUN #5 TCR=93% SCR=88% RQD=88%
			6	RUN			258			0 0 4 >10 3	RUN #6 TCR=100% SCR=80% RQD=63%
257.2 16.3	END OF BOREHOLE AT 16.3m. BOREHOLE GROUTED TO SURFACE.										

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

20 15-⊕-5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No P1-03

1 OF 1 **METRIC**

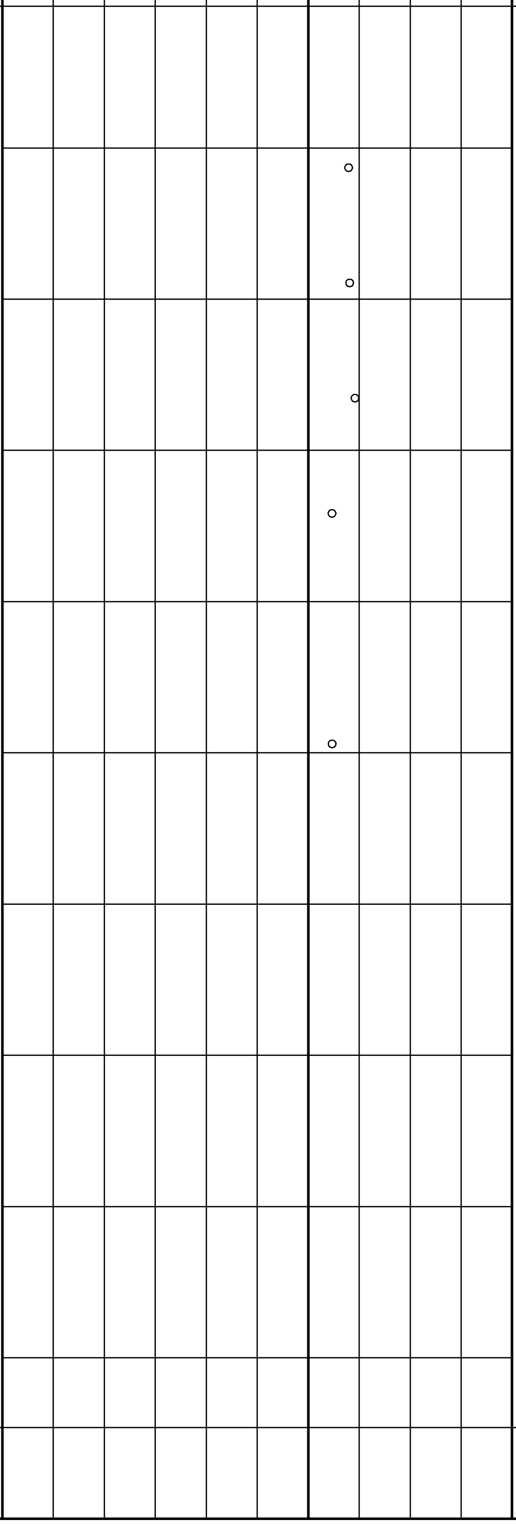
W.P. 3080-12-02/03 LOCATION MTM NAD 83 Zone 10: N 4 806 785.0 E 232 955.6 ORIGINATED BY AF
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2018.04.06 - 2018.04.09 LATITUDE 43.397526 LONGITUDE -80.386909 CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
272.9	GROUND SURFACE						
0.0	**ORGANICS/TOPSOIL** Soft Grey Wet						
271.7		1	SS	5	272		
1.2	**SAND** and **GRAVEL**, some silt, occasional organics, cobbles and boulders Dense Grey Wet	2	SS	44	271		29 59 12 (SI+CL)
	Very Loose	3	SS	1	270		
	Compact	4	SS	15	269		
268.4							
4.6	Silty **SAND**, some gravel, trace clay Very Dense Grey Moist (TILL)	5	SS	53	268		15 50 25 10
		6	SS	100/ 0.150	267		
266.5							
6.4	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy	1	RUN		266 265	FI >10 >10 4	RUN #1 TCR=47% SCR=15% RQD=0%
		2	RUN		264	>10 >10 5 0 0	RUN #2 TCR=86% SCR=67% RQD=26%
263.5							
9.4	END OF BOREHOLE AT 9.4m. BOREHOLE GROUTED TO SURFACE.						

GROUND WATER CONDITIONS

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

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



+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No P1-04

1 OF 2

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 762.0 E 232 960.9
ORIGINATED BY AF
DIST HWY 401
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2018.04.09 - 2018.04.10
LATITUDE 43.397319
LONGITUDE -80.386840
CHECKED BY MRA

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
273.2	GROUND SURFACE
0.0	**ORGANICS/TOPSOIL:** (600mm)
272.6 0.6	Silty **SAND**, some gravel, occasional roots, organics, cobbles and boulders Compact to Loose Brown Wet
268.6 4.6	Silty **SAND**, some gravel, trace clay Dense Grey Moist (TILL)
267.0 6.2	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy

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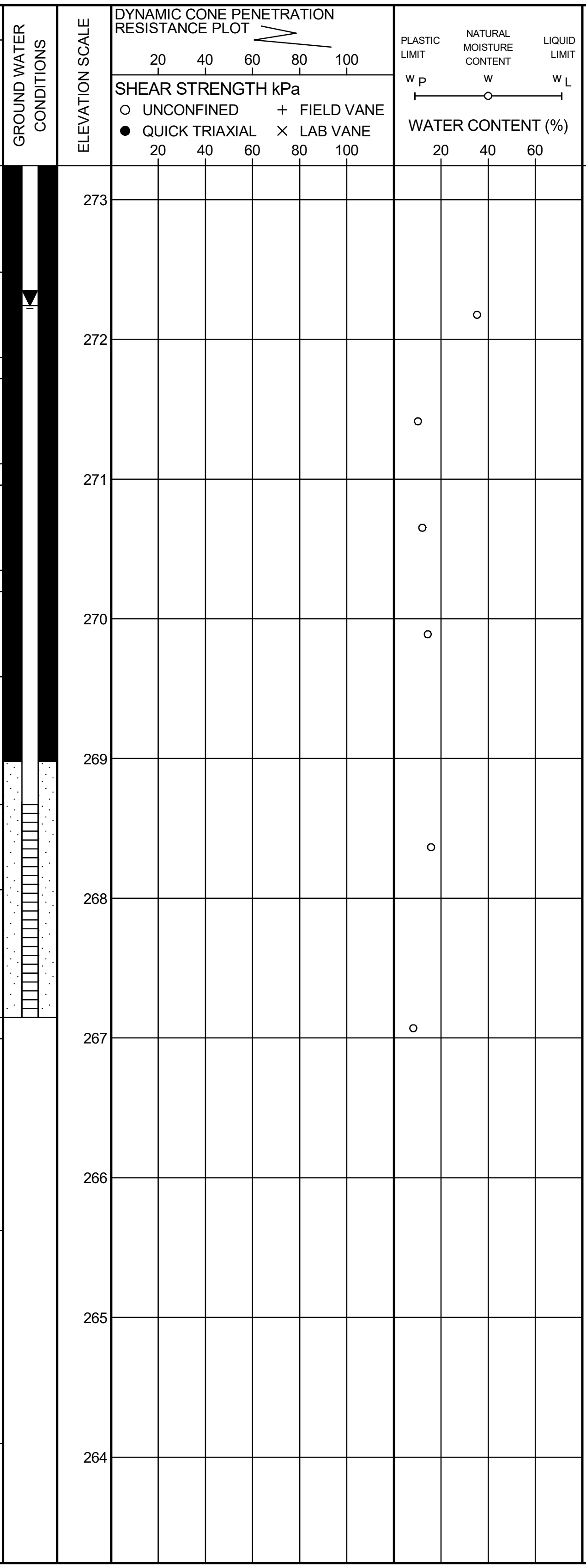
SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	27		
2	SS	10		
3	SS	9		17 53 21 9
4	SS	11		
5	SS	40		
6	SS	100/ 0.050	FI	
1	RUN		>10 >10 >10 >10 5	RUN #1 TCR=85% SCR=8% RQD=0%
2	RUN		>10 >10 >10 >10	RUN #2 TCR=82% SCR=28% RQD=0%
3	RUN		>10 >10	RUN #3 TCR=76% SCR=5% RQD=0%



+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No P1-04

2 OF 2

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 762.0 E 232 960.9
ORIGINATED BY AF
DIST HWY 401
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2018.04.09 - 2018.04.10
LATITUDE 43.397319
LONGITUDE -80.386840
CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
										>10	
							263			>10	
										>10	
			4	RUN			262			>10	RUN #4 TCR=29% SCR=3% RQD=0%
										>10	
	slightly weathered to fresh, fair to good quality						261			3	
										1	RUN #5 TCR=100% SCR=98% RQD=88%
			5	RUN						0	
							260			0	
										0	
										0	
			6	RUN			259			1	RUN #6 TCR=96% SCR=94% RQD=76%
										0	
258.4										0	
14.8	END OF BOREHOLE AT 14.8m. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.										

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.04.10	0.6	272.6
2018.04.11	0.8	272.4
2018.10.22	1.0	272.2

ONTMT4S2 MTO-11373.GPJ 2017TEMPLATE(MTO).GDT 11/21/18

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No P2-01

1 OF 2

METRIC

W.P. 3080-12-02/03	LOCATION MTM NAD 83 Zone 10: N 4 806 857.1 E 232 994.5		ORIGINATED BY BL
DIST	HWY 401	BOREHOLE TYPE HW Casing/NQ Coring	COMPILED BY MP
DATUM Geodetic	DATE 2018.10.21 - 2018.10.22	LATITUDE 43.398178 LONGITUDE -80.386438	CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT
272.1	GROUND SURFACE	
0.0	**SAND** and **GRAVEL**, trace to some silt, occasional to numerous cobbles and boulders Compact to Very Dense Grey Moist	
269.1 3.0	Silty **SAND**, some gravel, trace clay, occasional cobbles Dense to Very Dense Grey Moist (TILL)	
266.7 5.3	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy	

SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
				272	Depth of river water = 0.3m
1	SS	32		271	
2	SS	60			61 30 9 (SI+CL)
				270	
3	SS	21			
				269	
4	SS	33			10 54 28 8
				268	
5	SS	100 0.250		267	
1	RUN			266	RUN #1 TCR=50% SCR=17% RQD=0%
2	RUN			265	RUN #2 TCR=58% SCR=28% RQD=12%
				264	
3	RUN			263	RUN #3 TCR=55% SCR=43% RQD=25%
					RUN #4

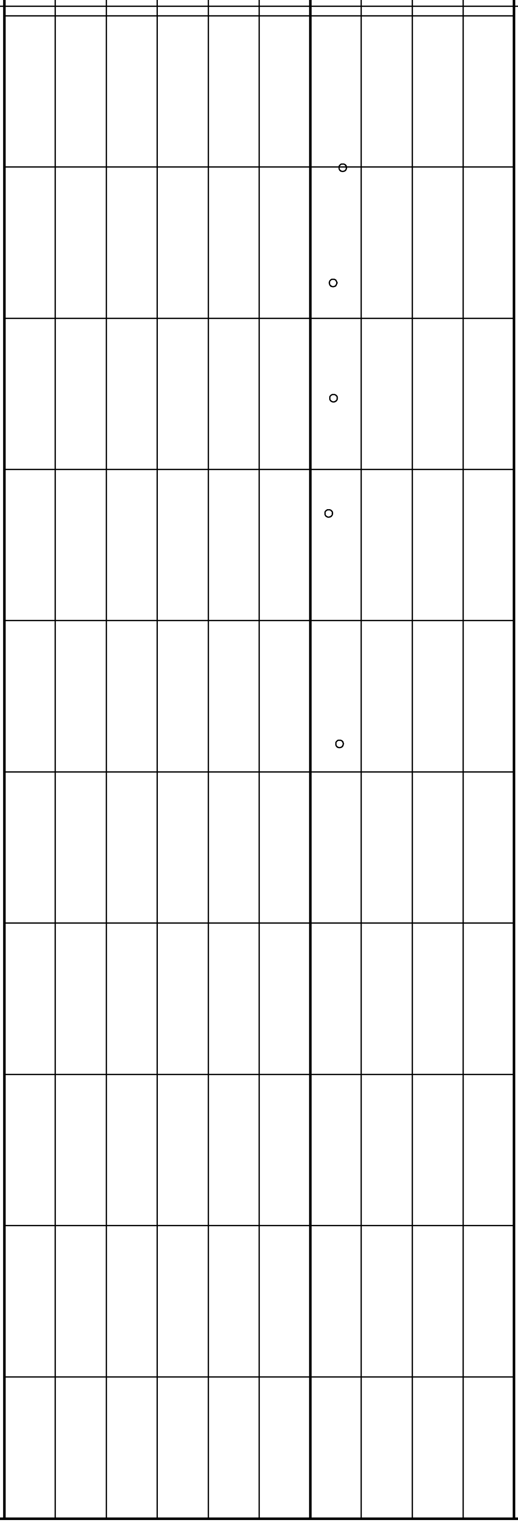
DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

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

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

WATER CONTENT (%)
20 40 60

UNIT WEIGHT γ kN/m3



FI
>10
>10
>10
>10
>10
>10
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>10
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>10
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>10
>10

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No P2-01

2 OF 2

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 857.1 E 232 994.5
ORIGINATED BY BL
DIST HWY 401
BOREHOLE TYPE HW Casing/NQ Coring
COMPILED BY MP
DATUM Geodetic
DATE 2018.10.21 - 2018.10.22
LATITUDE 43.398178
LONGITUDE -80.386438
CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	highly weathered to fresh, poor to fair quality	▨					262			5	TCR=100% SCR=95% RQD=73%
		▨								>10	
		▨	4	RUN						0	
		▨								2	
		▨					261				
		▨								>10	
		▨								4	RUN #5 TCR=100% SCR=98% RQD=33%
		▨								4	
		▨					260				
		▨	5	RUN						4	
		▨								>5	
		▨								>5	
		▨					259			4	RUN #6 TCR=100% SCR=100% RQD=87%
		▨								0	
		▨	6	RUN							
		▨								1	
		▨					258				
		▨								2	
257.6		▨								1	
14.5	END OF BOREHOLE AT 14.5m. BOREHOLE BACKFILLED WITH BENTONITE TO SURFACE.										

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

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No P2-02

1 OF 1 **METRIC**

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 843.5 E 232 996.9
ORIGINATED BY BL
DIST HWY 401
BOREHOLE TYPE HW Casing/NQ Coring
COMPILED BY MP
DATUM Geodetic
DATE 2018.10.21 - 2018.10.21
LATITUDE 43.398056
LONGITUDE -80.386407
CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
272.0	GROUND SURFACE								
0.0	**SAND** and **GRAVEL**, trace to some silt, occasional to numerous cobbles and boulders Dense to Very Dense Grey Wet					272			Depth of river water = 0.3m
		1	SS	37		271	○		
		2	SS	100/ 0.300		270			
	Compact	3	SS	14			○		
		4	SS	23		269	○		
		5	SS	100/ 0.275		268	○		62 30 8 (SI+CL)
267.1 4.9	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy	1	RUN			267		FI >10 >10	RUN #1 TCR=67% SCR=29% RQD=21%
		2	RUN			266		>10 >10 >10 >10	RUN #2 TCR=50% SCR=32% RQD=0%
		3	RUN			265 264		>10 >10 >10 >10 >10 >10	RUN #3 TCR=58% SCR=38% RQD=7%
263.5 8.6	END OF BOREHOLE AT 8.6m. BOREHOLE BACKFILLED WITH BENTONITE TO SURFACE.								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%): 20 40 60

+ ³, × ³: Numbers refer to Sensitivity

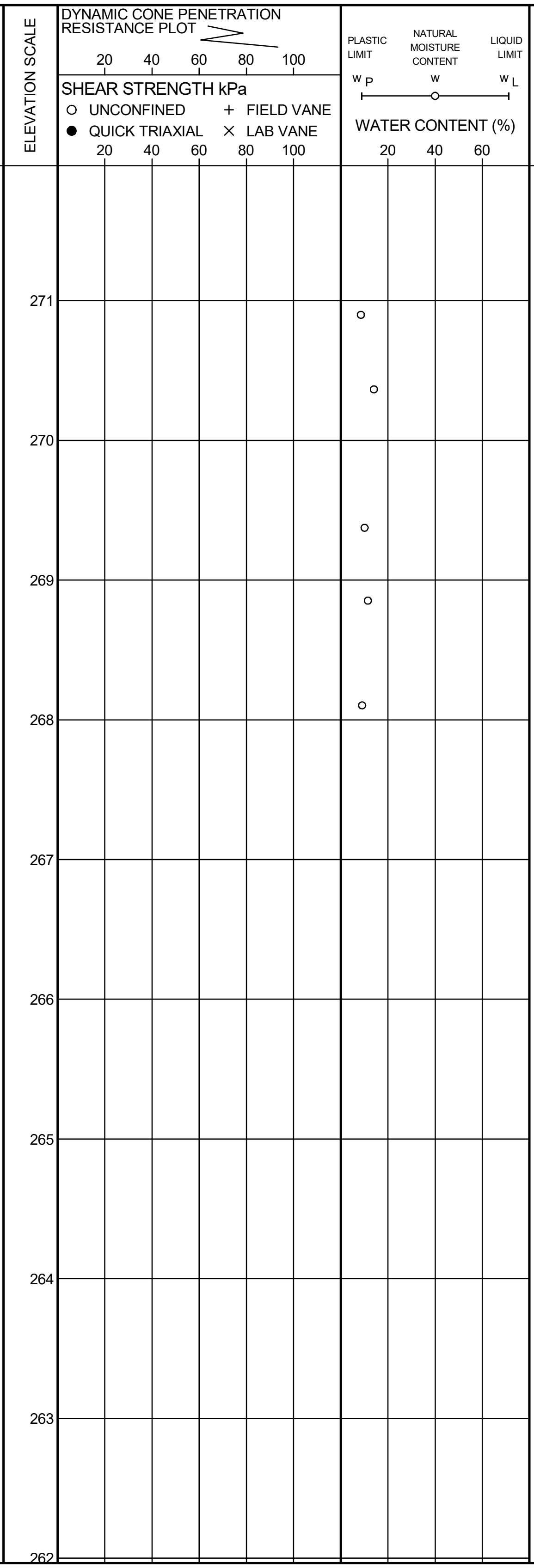
20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No P2-03

1 OF 2 METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 817.6 E 232 998.9
ORIGINATED BY BL
DIST HWY 401
BOREHOLE TYPE HW Casing/NQ Coring
COMPILED BY MP
DATUM Geodetic
DATE 2018.10.19 - 2018.10.20
LATITUDE 43.397823
LONGITUDE -80.386379
CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
272.0	GROUND SURFACE					
0.0	**SAND** and **GRAVEL**, trace to some silt, trace clay, occasional to numerous cobbles and boulders Very Dense Brown Moist					Depth of river water = 0.3m
		1	SS	100/ 0.300		
		2	SS	100/ 0.150		
269.7 2.3	Gravelly **SAND**, some silt, trace clay, occasional cobbles and boulders Very Dense Brown Moist	3	SS	74		23 50 17 10
		4	SS	100/ 0.125		
268.2 3.8	Silty **SAND**, some gravel, occasional cobbles Very Dense Grey Wet (TILL)	5	SS	100/ 0.125		
267.3 4.7	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy	1	RUN		FI >10 >10 1	RUN #1 TCR=45% SCR=17% RQD=8%
		2	RUN		>10 >10 >10	RUN #2 TCR=52% SCR=22% RQD=7%
		3	RUN		>10 >10 >5	RUN #3 TCR=52% SCR=37% RQD=7%
		4	RUN		>10 >10 1	RUN #4 TCR=48% SCR=28% RQD=17%
	slightly weathered to fresh, poor to excellent quality				2	RUN #5 TCR=100% SCR=85%



STRAT PLOT · GROUND WATER CONDITIONS

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No P2-03

2 OF 2

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 817.6 E 232 998.9
ORIGINATED BY BL
DIST
HWY 401
BOREHOLE TYPE HW Casing/NQ Coring
COMPILED BY MP
DATUM Geodetic
DATE 2018.10.19 - 2018.10.20
LATITUDE 43.397823
LONGITUDE -80.386379
CHECKED BY MRA

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
										5	RQD=37%
			5	RUN						>10	
										>10	
							261			2	RUN #6 TCR=80% SCR=80% RQD=30%
										5	
			6	RUN						5	
							260			4	
										5	
										0	RUN #7 TCR=93% SCR=93% RQD=55%
							259			2	
										2	
			7	RUN						3	
							258			>10	
257.8										>10	
14.2	END OF BOREHOLE AT 14.2m. BOREHOLE BACKFILLED WITH BENTONITE TO SURFACE.										

+³, ×³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

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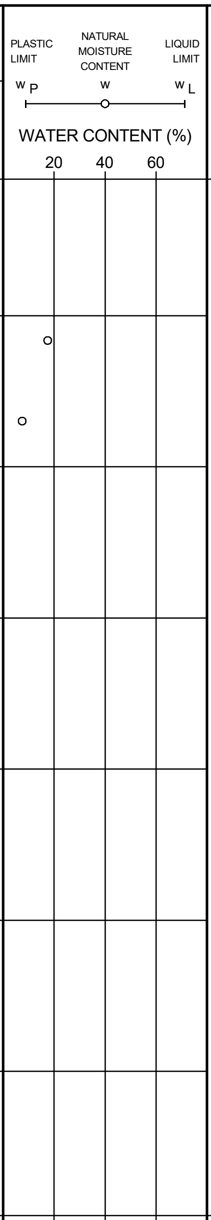
RECORD OF BOREHOLE No P2-04

1 OF 1 **METRIC**

W.P.	3080-12-02/03	LOCATION	MTM NAD 83 Zone 10: N 4 806 790.3 E 233 000.9	ORIGINATED BY	BL
DIST	HWY 401	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	MP
DATUM	Geodetic	DATE	2018.10.19 - 2018.10.19 LATITUDE 43.397577 LONGITUDE -80.386350	CHECKED BY	MRA



ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
271.9	GROUND SURFACE							
0.0	**SAND**, some gravel, some silt, occasional to numerous cobbles and boulders Very Dense Grey Moist							Depth of river water = 0.3m
		1	SS	100/ 0.275		271		17 60 18 5
270.5 1.4	Silty **SAND**, some gravel, trace clay, occasional cobbles and boulders Very Dense Grey Wet (TILL)	2	SS	100/ 0.150				
		1	RUN			270		
268.9 3.0	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy	2	RUN			269	FI >10 >10	RUN #2 TCR=45% SCR=15% RQD=0%
		3	RUN			268 267	0 >5 >10 >10 >10	RUN #3 TCR=72% SCR=37% RQD=15%
		4	RUN			266	0 >10 >10 >10 0 0	RUN #4 TCR=47% SCR=10% RQD=0%
265.0 6.9	END OF BOREHOLE AT 6.9m. BOREHOLE BACKFILLED WITH BENTONITE TO SURFACE.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

+³, ×³: Numbers refer to Sensitivity

20
15-⊕-5 (%) STRAIN AT FAILURE
10

Ministry of Transportation
Ontario

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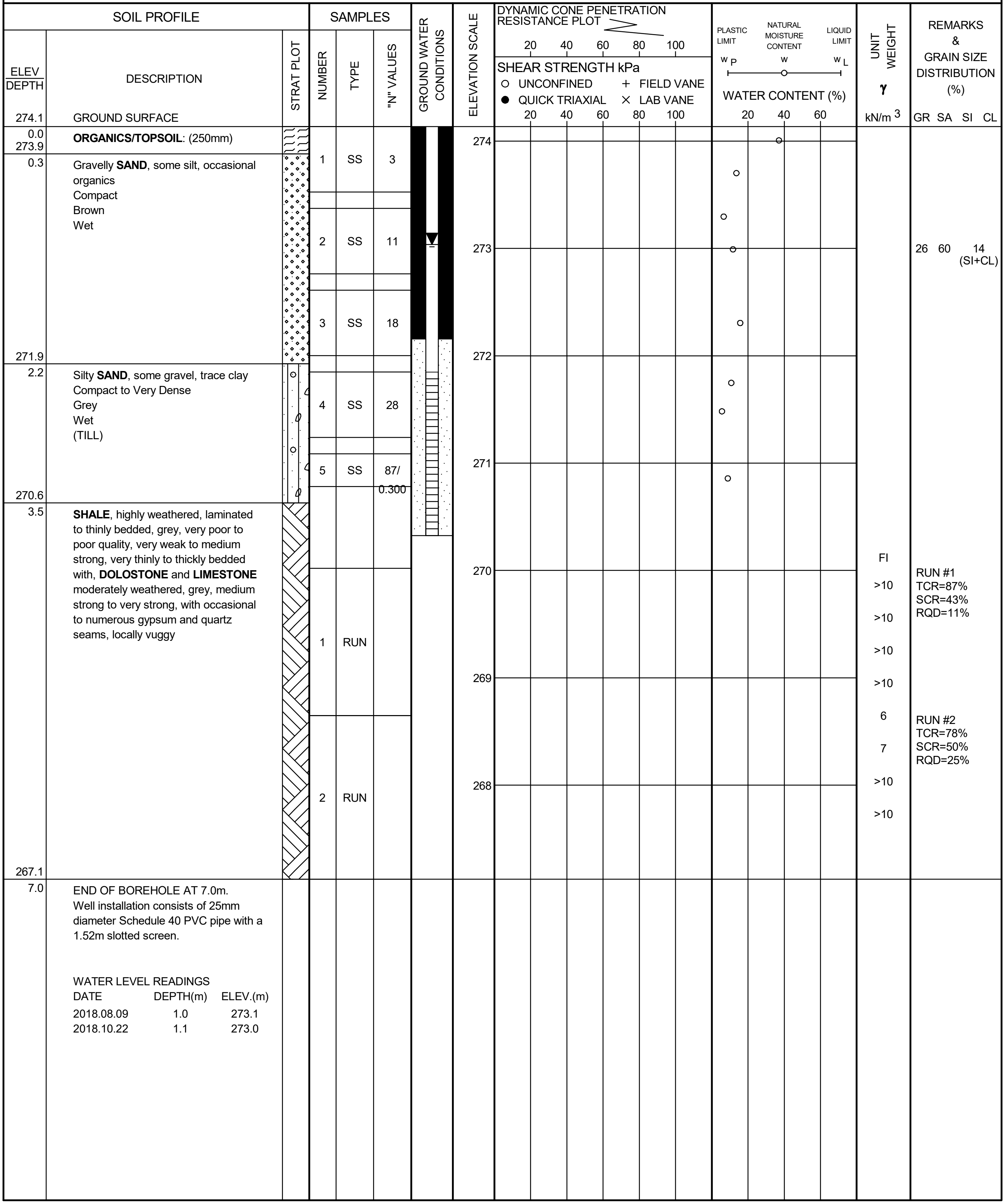
RECORD OF BOREHOLE No P3-01

1 OF 1 **METRIC**

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 898.1 E 233 066.0
ORIGINATED BY JM
DIST ____ HWY 401
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY MP
DATUM Geodetic
DATE 2018.07.23 - 2018.07.24
LATITUDE 43.398554
LONGITUDE -80.385561
CHECKED BY MRA



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
274.1	GROUND SURFACE					
0.0 273.9	**ORGANICS/TOPSOIL**: (250mm)					
0.3	Gravelly **SAND**, some silt, occasional organics Compact Brown Wet	1	SS	3		
		2	SS	11		26 60 14 (SI+CL)
271.9		3	SS	18		
2.2	Silty **SAND**, some gravel, trace clay Compact to Very Dense Grey Wet (TILL)	4	SS	28		
270.6		5	SS	87/ 0.300		
3.5	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy	1	RUN		FI >10 >10 >10 >10	RUN #1 TCR=87% SCR=43% RQD=11%
267.1		2	RUN		6 7 >10 >10	RUN #2 TCR=78% SCR=50% RQD=25%
7.0	END OF BOREHOLE AT 7.0m. Well installation consists of 25mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.					

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.08.09	1.0	273.1
2018.10.22	1.1	273.0

+3, ×3: Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No P3-02

1 OF 2

METRIC

W.P. 3080-12-02/03	LOCATION MTM NAD 83 Zone 10: N 4 806 892.6 E 233 065.8		ORIGINATED BY JM
DIST ____ HWY 401	BOREHOLE TYPE Hollow Stem Augers/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.07.24 - 2018.07.24	LATITUDE 43.398504 LONGITUDE -80.385562	CHECKED BY MRA

SOIL PROFILE

ELEV DEPTH	DESCRIPTION	STRAT PLOT
273.6	GROUND SURFACE	
0.0 273.3	**ORGANICS/TOPSOIL:** (250mm)	
0.3	Silty **SAND**, some gravel, occasional to numerous organics Very Loose Grey Moist	
272.5 1.1	**PEAT:** (300mm) Brown	
272.2 1.4	Gravelly Silty **SAND**, trace clay Dense to Very Dense Grey Wet	
270.0 3.6	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy	

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	2			
2	SS	37			
3	SS	38			
4	SS	77			26 43 31 (SI+CL)
5	SS	66			
1	RUN			FI >10 >10 >10	RUN #1 TCR=74% SCR=57% RQD=26%
2	RUN			>10 >10 7 >10	RUN #2 TCR=65% SCR=43% RQD=0%
3	RUN			10 >10 >10 >10 >10	RUN #3 TCR=85% SCR=65% RQD=0%
4	RUN			>10 6 7	RUN #4 TCR=63% SCR=53% RQD=17%

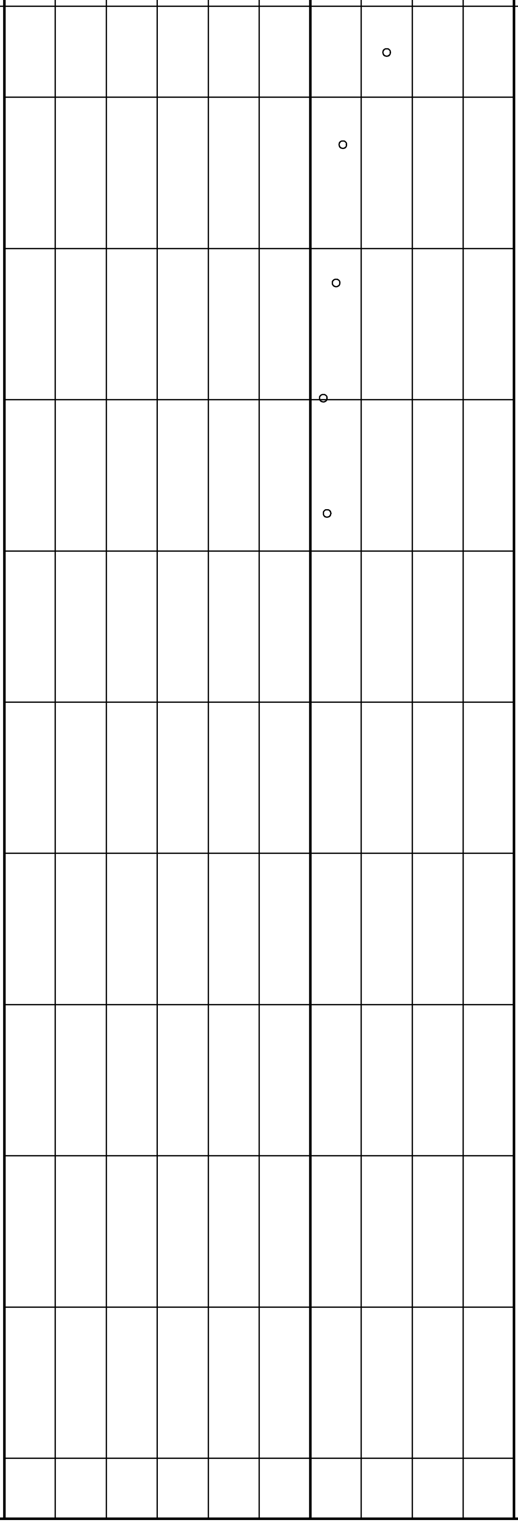
ELEVATION SCALE: 273, 272, 271, 270, 269, 268, 267, 266, 265, 264

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%) 20 40 60



+ 3, × 3: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

ONTMT4S2 MTO-11373.GPJ 2017TEMPLATE(MTO).GDT 11/21/18

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No P3-02

2 OF 2

METRIC

W.P.	3080-12-02/03	LOCATION	MTM NAD 83 Zone 10: N 4 806 892.6 E 233 065.8	ORIGINATED BY	JM
DIST	HWY 401	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	MP
DATUM	Geodetic	DATE	2018.07.24 - 2018.07.24 LATITUDE 43.398504 LONGITUDE -80.385562	CHECKED BY	MRA

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	slightly weathered to fresh, fair quality										RUN #5 TCR=98% SCR=98% RQD=54%
										10	
							263			5	
			5	RUN						3	
										5	
							262			5	RUN #6 TCR=100% SCR=100% RQD=66%
										7	
										4	
			6	RUN						3	
							261			3	
										1	
	excellent quality									1	RUN #7 TCR=100% SCR=100% RQD=100%
							260			1	
			7	RUN						0	
259.3										1	
14.3	END OF BOREHOLE AT 14.3m. BOREHOLE BACKFILLED WITH BENTONITE TO SURFACE.										

+ 3, × 3 : Numbers refer to Sensitivity

20
15 ⊕ 5 (%) STRAIN AT FAILURE
10

Ministry of Transportation
Ontario

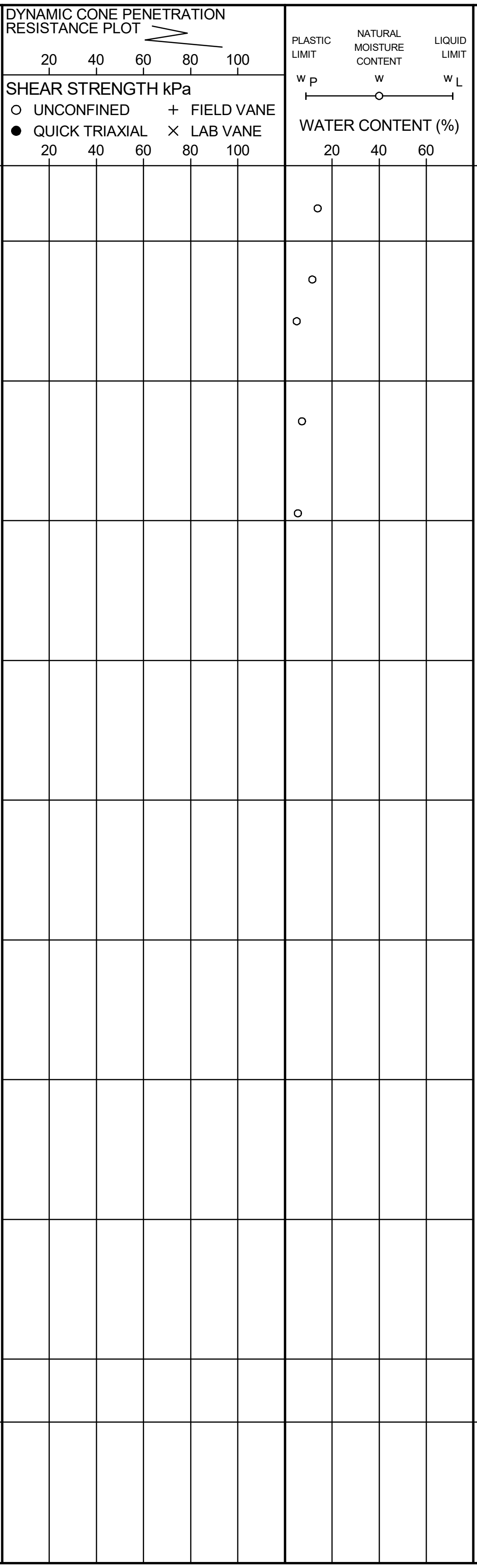
THURBER

RECORD OF BOREHOLE No P3-03

1 OF 1 **METRIC**

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 867.4 E 233 076.1
ORIGINATED BY JM
DIST
HWY 401
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY MP
DATUM Geodetic
DATE 2018.07.25 - 2018.07.25
LATITUDE 43.398279
LONGITUDE -80.385432
CHECKED BY MRA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
272.5	GROUND SURFACE							
0.0	**SAND** and **GRAVEL**, some silt, occasional organics Very Loose Brown Moist	1	SS	2		272		
271.7 0.9	**SAND** and **GRAVEL**, some silt Compact to Very Dense Grey Wet	2	SS	27		271		
		3	SS	84				52 33 15 (SI+CL)
		4	SS	89/ 0.250		270		
269.8 2.7	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy					269	FI	
		1	RUN				>10 >10	RUN #1 TCR=63% SCR=13% RQD=0%
		2	RUN			268	>10 >10 >10	RUN #2 TCR=58% SCR=23% RQD=0%
		3	RUN			267 266	>10	RUN #3 TCR=20% SCR=0% RQD=0%
		4	RUN			265 264	>10 5 9 4	RUN #4 TCR=68% SCR=63% RQD=13%
263.5 9.0	END OF BOREHOLE AT 9.0m. BOREHOLE BACKFILLED WITH BENTONITE TO SURFACE.							



+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No P3-04

1 OF 2 METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 850.2 E 233 078.8
ORIGINATED BY JM
DIST
HWY 401
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY MP
DATUM Geodetic
DATE 2018.07.26 - 2018.07.26
LATITUDE 43.398124
LONGITUDE -80.385397
CHECKED BY MRA

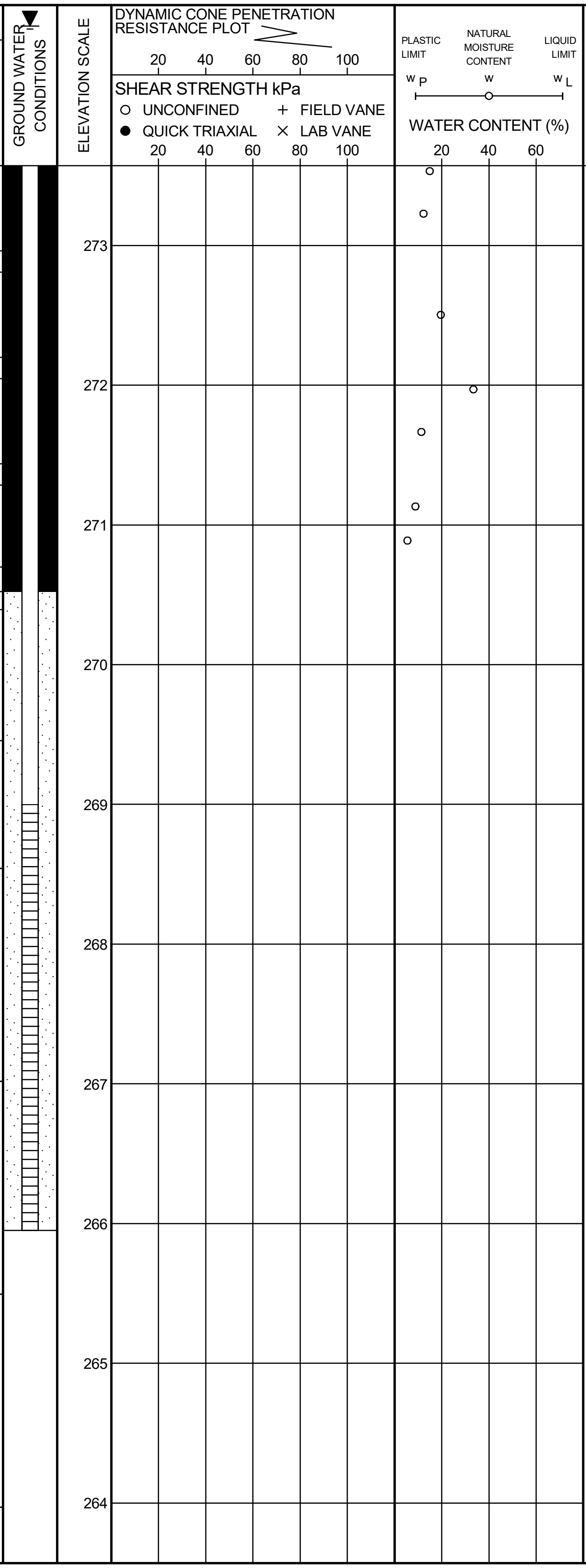
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
273.6	GROUND SURFACE
0.0 0.1	**ORGANICS/TOPSOIL**: (75mm)
	SAND, some gravel to gravelly, some silt, trace clay, occasional organics, cobbles and boulders Compact to Loose Brown Moist
271.4 2.1	Silty **SAND**, some gravel Very Dense Grey Wet (TILL)
270.6 3.0	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy

SAMPLES

NUMBER	TYPE	"N" VALUES	WATER CONTENT / REMARKS
1	SS	14	
2	SS	5	
3	SS	6	GR SA SI CL: 29 44 20 7
4	SS	79	
5	SS	50/ 0.125	
1	RUN		RUN #1 TCR=100% SCR=33% RQD=11%
2	RUN		RUN #2 TCR=55% SCR=25% RQD=7%
3	RUN		RUN #3 TCR=42% SCR=42% RQD=10%
4	RUN		RUN #4 TCR=87% SCR=83% RQD=23%
			RUN #5 TCR=98% SCR=95%

UNIT WEIGHT γ kN/m³ column: FI >10, >10, >10, >10, 8, >10, 5, 8, >10, 5, 7, 7, 4, 6



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+ ³, × ³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-11373.GPJ 2017TEMPLATE(MTO).GDT 11/21/18

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No P3-04

2 OF 2

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 850.2 E 233 078.8
ORIGINATED BY JM
DIST
HWY 401
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY MP
DATUM Geodetic
DATE 2018.07.26 - 2018.07.26
LATITUDE 43.398124
LONGITUDE -80.385397
CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
		▨								>10	RQD=37%
		▨	5	RUN						6	
		▨					263				
		▨								7	RUN #6 TCR=98% SCR=98% RQD=40%
		▨					262			5	
		▨	6	RUN						5	
		▨								3	
		▨								3	
		▨					261				
	excellent quality	▨								6	RUN #7 TCR=100% SCR=100% RQD=92%
		▨								2	
		▨	7	RUN						3	
		▨					260				
		▨								2	
259.4 14.2	END OF BOREHOLE AT 14.2m. Well installation consists of 50mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen.									1	

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.10.22	-1.0	274.6
artesian		

+ ³, × ³: Numbers refer to Sensitivity

20
15 ⊕ 5 (%) STRAIN AT FAILURE
10

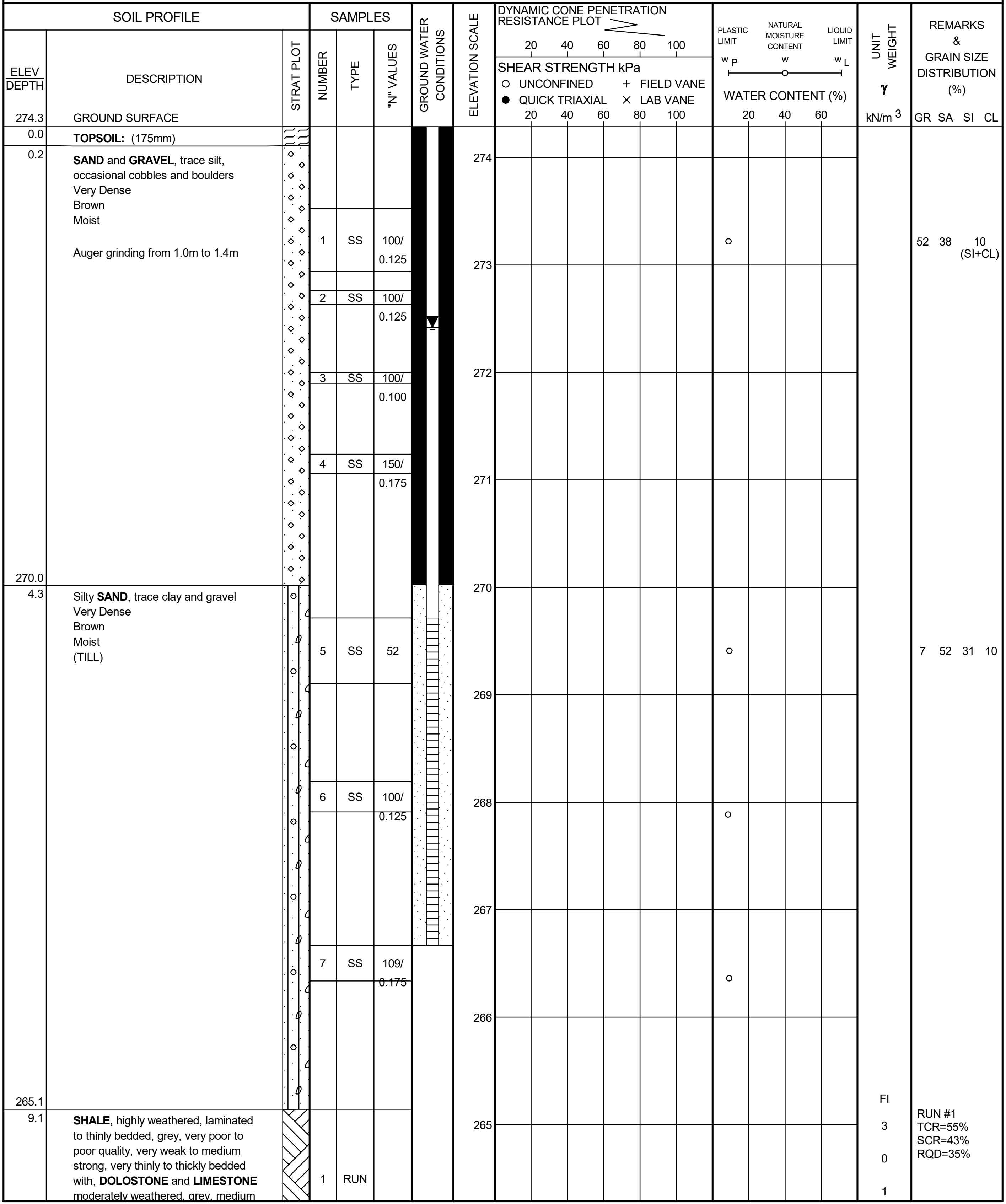
Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No WA-01

1 OF 2 **METRIC**

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 792.0 E 232 913.1
ORIGINATED BY AF
DIST HWY 401
BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2018.04.03 - 2018.04.12
LATITUDE 43.397584
LONGITUDE -80.387435
CHECKED BY MRA



ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
274.3	GROUND SURFACE								
0.0	**TOPSOIL:** (175mm)								
0.2	**SAND** and **GRAVEL**, trace silt, occasional cobbles and boulders Very Dense Brown Moist Auger grinding from 1.0m to 1.4m	1	SS	100/ 0.125		52	38	10 (SI+CL)	
		2	SS	100/ 0.125					
		3	SS	100/ 0.100					
		4	SS	150/ 0.175					
270.0 4.3	Silty **SAND**, trace clay and gravel Very Dense Brown Moist (TILL)	5	SS	52		7	52	31	10
		6	SS	100/ 0.125					
		7	SS	109/ 0.175					
265.1 9.1	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium	1	RUN		FI 3 0 1	RUN #1 TCR=55% SCR=43% RQD=35%			

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No WA-01

2 OF 2

METRIC

W.P. 3080-12-02/03	LOCATION MTM NAD 83 Zone 10: N 4 806 792.0 E 232 913.1		ORIGINATED BY AF
DIST HWY 401	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring		COMPILED BY AN
DATUM Geodetic	DATE 2018.04.03 - 2018.04.12	LATITUDE 43.397584 LONGITUDE -80.387435	CHECKED BY MRA

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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS
	Continued From Previous Page						
	strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy clay seam (125mm) at 9.3m				264		
		2	RUN			>10 >10 2	RUN #2 TCR=61% SCR=14% RQD=7%
					263		
		3	RUN		262	>10 >10 >10 >10 4	RUN #3 TCR=95% SCR=13% RQD=0%
					261		
	slightly weathered to fresh, excellent quality	4	RUN		260	0 0 0 0 0	RUN #4 TCR=95% SCR=95% RQD=95%
259.4 14.9	END OF BOREHOLE AT 14.9m. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen.						

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.04.13	1.7	272.6
2018.08.09	2.0	272.3
2018.10.22	1.9	272.4

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

$\overset{20}{15\text{-}\oplus\text{-}5}$ 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No WA-02

1 OF 2 **METRIC**

W.P. 3080-12-02/03	LOCATION MTM NAD 83 Zone 10: N 4 806 767.2 E 232 930.8		ORIGINATED BY AG
DIST HWY 401	BOREHOLE TYPE NW Casing/NQ Coring		COMPILED BY AN
DATUM Geodetic	DATE 2018.04.03 - 2018.04.12	LATITUDE 43.397363 LONGITUDE -80.387212	CHECKED BY MRA

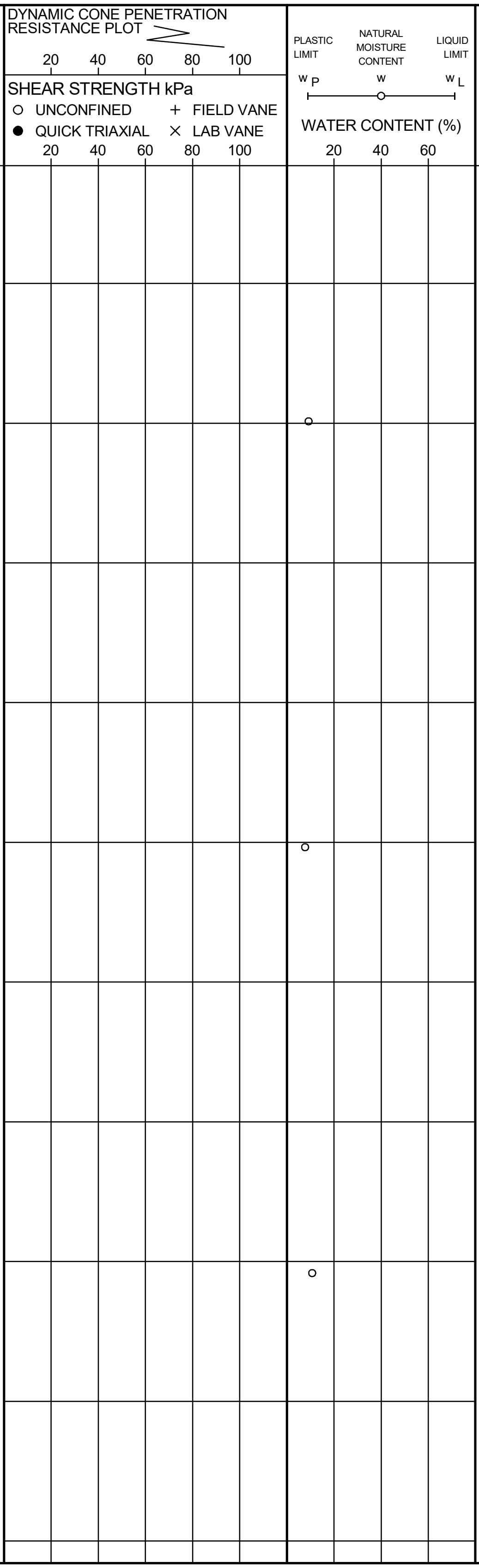
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
273.8	GROUND SURFACE
0.0	**TOPSOIL:** (150mm)
0.2	**SAND** and **GRAVEL**, trace to some silt, occasional to numerous cobbles and boulders Dense to Very Dense Brown Moist
269.9 4.0	Silty **SAND**, some clay, trace gravel, occasional cobbles and boulders Very Dense Light Brown Moist (TILL)
265.6 8.2	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy Highly fractured zone from 8.2m to 8.8m

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	35		
2	SS	100/ 0.025		
3	SS	55		
4	SS	100/ 0.250		10 43 34 13
5	SS	108/ 0.175		
6	SS	100/ 0.275		
1	RUN		FI >10 >10 2	RUN #1 TCR=60% SCR=8% RQD=7%

ELEVATION SCALE: 273, 272, 271, 270, 269, 268, 267, 266, 265, 264



Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No WA-02

2 OF 2

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 767.2 E 232 930.8
ORIGINATED BY AG
DIST ______ HWY 401
BOREHOLE TYPE NW Casing/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2018.04.03 - 2018.04.12
LATITUDE 43.397363
LONGITUDE -80.387212
CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m ^{3}	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
262.9		▨	2	RUN			263			8 3 2	RUN #2 TCR=71% SCR=38% RQD=15%
11.0	END OF BOREHOLE AT 11.0m. BOREHOLE GROUTED TO SURFACE.										

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

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

RECORD OF BOREHOLE No WA-03

1 OF 2

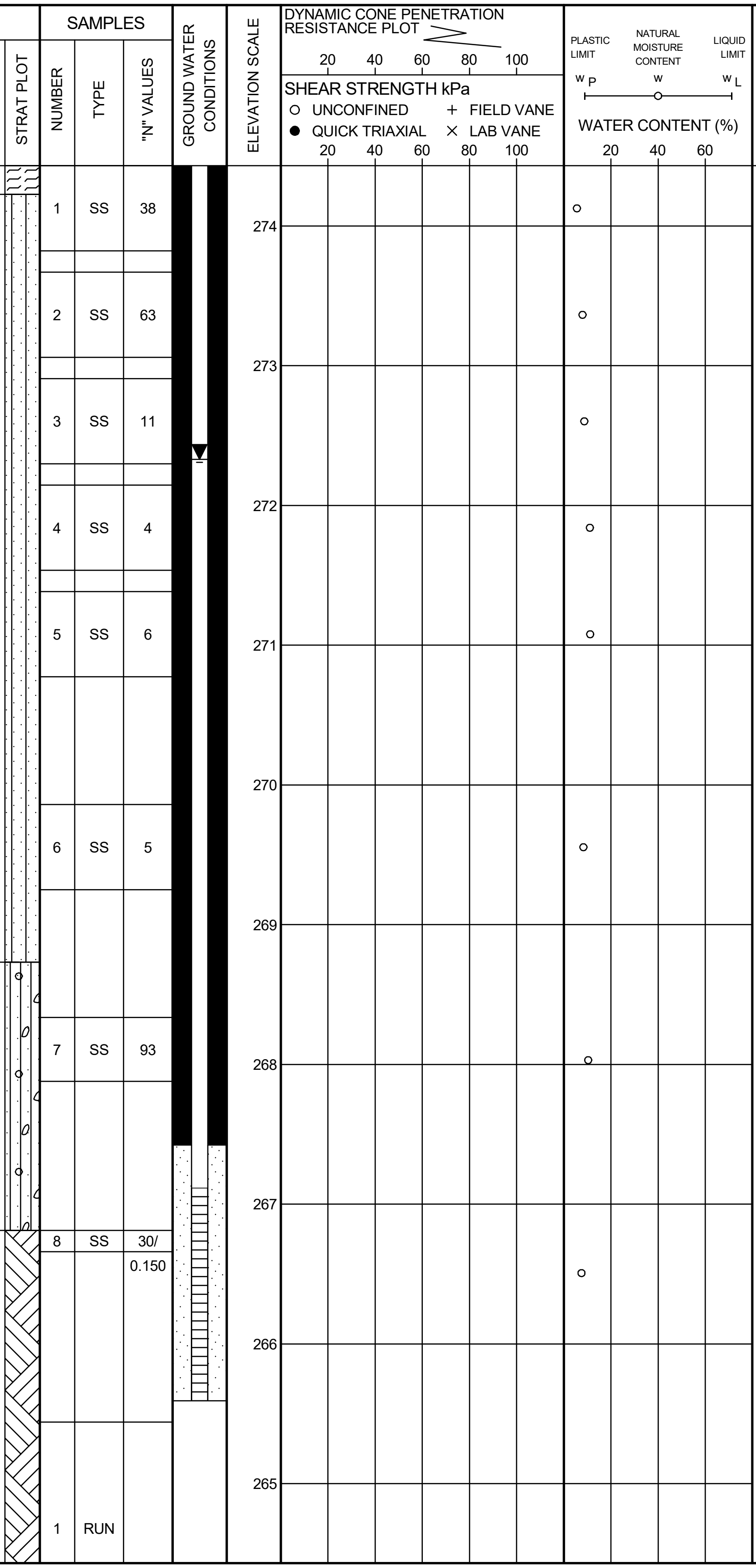
METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 739.4 E 232 931.7
ORIGINATED BY GA/AF
DIST HWY 401
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2018.03.29 - 2018.04.02
LATITUDE 43.397113
LONGITUDE -80.387198
CHECKED BY MRA

ELEV DEPTH	DESCRIPTION
274.4	GROUND SURFACE
0.0	**TOPSOIL:** (200mm)
0.2	Silty **SAND**, some gravel, occasional rootlets Dense to Very Dense Brown Moist Loose
268.7 5.7	Sandy **SILT**, some clay Very Dense Brown Wet (TILL)
266.8 7.6	**SHALE**, highly weathered, laminated to thinly bedded, grey, very poor to poor quality, very weak to medium strong, very thinly to thickly bedded with, **DOLOSTONE** and **LIMESTONE** moderately weathered, grey, medium strong to very strong, with occasional to numerous gypsum and quartz seams, locally vuggy

Continued Next Page

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	38		
2	SS	63		
3	SS	11		17 53 23 7
4	SS	4		
5	SS	6		
6	SS	5		
7	SS	93		0 37 47 16
8	SS	30/ 0.150		
1	RUN		FI >10 >10 >10	RUN #1 TCR=50% SCR=0% RQD=0%



+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No WA-03

2 OF 2

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 739.4 E 232 931.7
ORIGINATED BY GA/AF
DIST HWY 401
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2018.03.29 - 2018.04.02
LATITUDE 43.397113
LONGITUDE -80.387198
CHECKED BY MRA

SOIL PROFILE · SAMPLES · DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) · SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100) · PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L · WATER CONTENT (%) (20 40 60)

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page							
							>10	
						264		
							>10	RUN #2 TCR=55% SCR=50% RQD=0%
							4	
							4	
		2	RUN					
						263		
	Slightly weathered to fresh, good to excellent quality							RUN #3 TCR=100% SCR=100% RQD=100%
							0	
						262		
							0	
		3	RUN					
							0	
							0	
						261		RUN #4 TCR=100% SCR=100% RQD=100%
							0	
							2	
		4	RUN				1	
						260		
							2	
							2	
							0	RUN #5 TCR=100% SCR=100% RQD=87%
							1	
						259		
		5	RUN				3	
							5	
							1	
						258		RUN #6 TCR=100% SCR=97% RQD=83%
							0	
							0	
		6	RUN				2	
						257		
							4	
							2	
256.4 18.0	END OF BOREHOLE AT 18.0m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.							

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.04.02	1.0	273.4
2018.04.03	1.9	272.5
2018.04.05	1.6	272.8
2018.04.09	1.7	272.7
2018.04.11	1.8	272.6
2018.08.09	2.3	272.1
2018.10.22	2.1	272.3

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

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

Ministry of Transportation
Ontario

THURBER

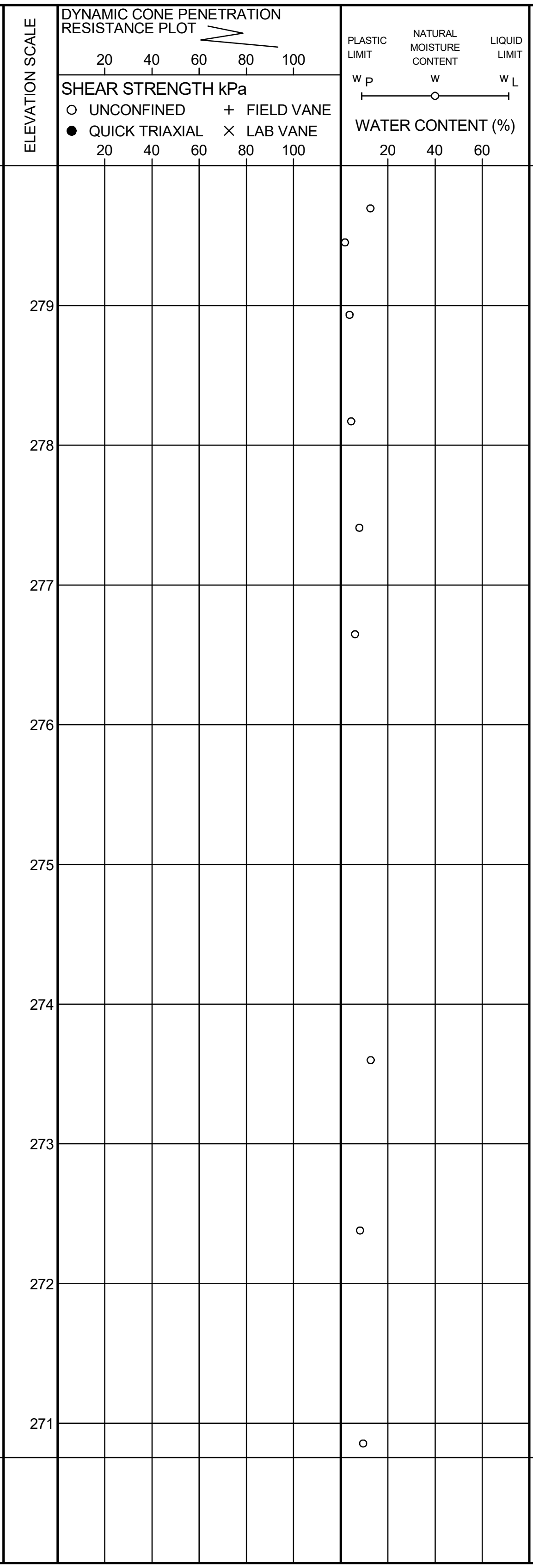
RECORD OF BOREHOLE No WA-04

1 OF 2

METRIC

W.P. 3080-12-02/03	LOCATION MTM NAD 83 Zone 10: N 4 806 765.6 E 232 903.9			ORIGINATED BY	RP
DIST	HWY 401	BOREHOLE TYPE Solid Stem Augers		COMPILED BY	MP
DATUM Geodetic	DATE 2018.04.30 - 2018.04.30	LATITUDE 43.397346	LONGITUDE -80.387545	CHECKED BY	MRA

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
280.0	GROUND SURFACE										
0.0 279.6	**ASPHALT:** (350mm)										
0.4	**SAND** and **GRAVEL**, trace silt Brown Moist (FILL)		1	SS	61						
			2	SS	13						
277.8 2.2	Silty **SAND**, some gravel, trace clay, occasional cobbles Compact to Very Dense Brown Moist (FILL)		3	SS	8			17	48	26	9
			4	SS	30						
			5	SS	60/ 0.225						
			6	SS	64						
272.8 7.2	**SAND** and **GRAVEL**, some silt, trace clay, occasional cobbles Very Dense Brown Moist		7	SS	90/ 0.250			33	44	18	5
			8	SS	50/ 0.100						
270.8 9.2	END OF BOREHOLE AT 9.2m. BOREHOLE OPEN TO 7.6m AND WATER LEVEL AT 7.5m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND										



REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No WA-04

2 OF 2

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 765.6 E 232 903.9
ORIGINATED BY RP
DIST HWY 401
BOREHOLE TYPE Solid Stem Augers
COMPILED BY MP
DATUM Geodetic
DATE 2018.04.30 - 2018.04.30
LATITUDE 43.397346
LONGITUDE -80.387545
CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	CUTTINGS TO 0.2m, THEN ASPHALT AT SURFACE.										

+ 3, × 3 : Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No WA-05

1 OF 2

METRIC

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 737.4 E 232 913.6
ORIGINATED BY SB
DIST HWY 401
BOREHOLE TYPE Solid Stem Augers
COMPILED BY MP
DATUM Geodetic
DATE 2017.12.11 - 2017.12.11
LATITUDE 43.397093
LONGITUDE -80.387420
CHECKED BY MRA

SOIL PROFILE			SAMPLES									REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	GR	SA	SI	CL
280.0	GROUND SURFACE													
0.0	**ASPHALT:** (200mm)													
0.2	**SAND** and **GRAVEL**, some silt Very Dense Brown Moist (FILL)		1	SS	78									
			2	SS	68		279							
	Loose to Compact		3	SS	16						37	49	14 (SI+CL)	
277.8							278							
2.2	Silty **SAND**, some gravel, trace clay, occasional cobbles Dense to Very Dense Brown Moist (FILL)		4	SS	8									
							277							
			5	SS	46									
							276							
			6	SS	92						18	49	25	8
							275							
							274							
			7	SS	5									
272.8							273							
7.2	**SAND** and **GRAVEL**, some silt, topsoil stained, occasional cobbles Dense to Very Dense Brown Wet													
			8	SS	40		272				53	36	11 (SI+CL)	
							271							
270.6			9	SS	100/ 0.300									
9.4	END OF BOREHOLE AT 9.5m. BOREHOLE OPEN TO 7.9m AND WATER LEVEL AT 7.3m.													

Continued Next Page

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

20
15 ⊕ 5 (%) STRAIN AT FAILURE
10

Ministry of
Transportation
Ontario

THURBER

RECORD OF BOREHOLE No WA-05

2 OF 2 **METRIC**

W.P. 3080-12-02/03
LOCATION MTM NAD 83 Zone 10: N 4 806 737.4 E 232 913.6
ORIGINATED BY SB
DIST
HWY 401
BOREHOLE TYPE Solid Stem Augers
COMPILED BY MP
DATUM Geodetic
DATE 2017.12.11 - 2017.12.11
LATITUDE 43.397093
LONGITUDE -80.387420
CHECKED BY MRA

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

+ ³, × ³: Numbers refer to Sensitivity

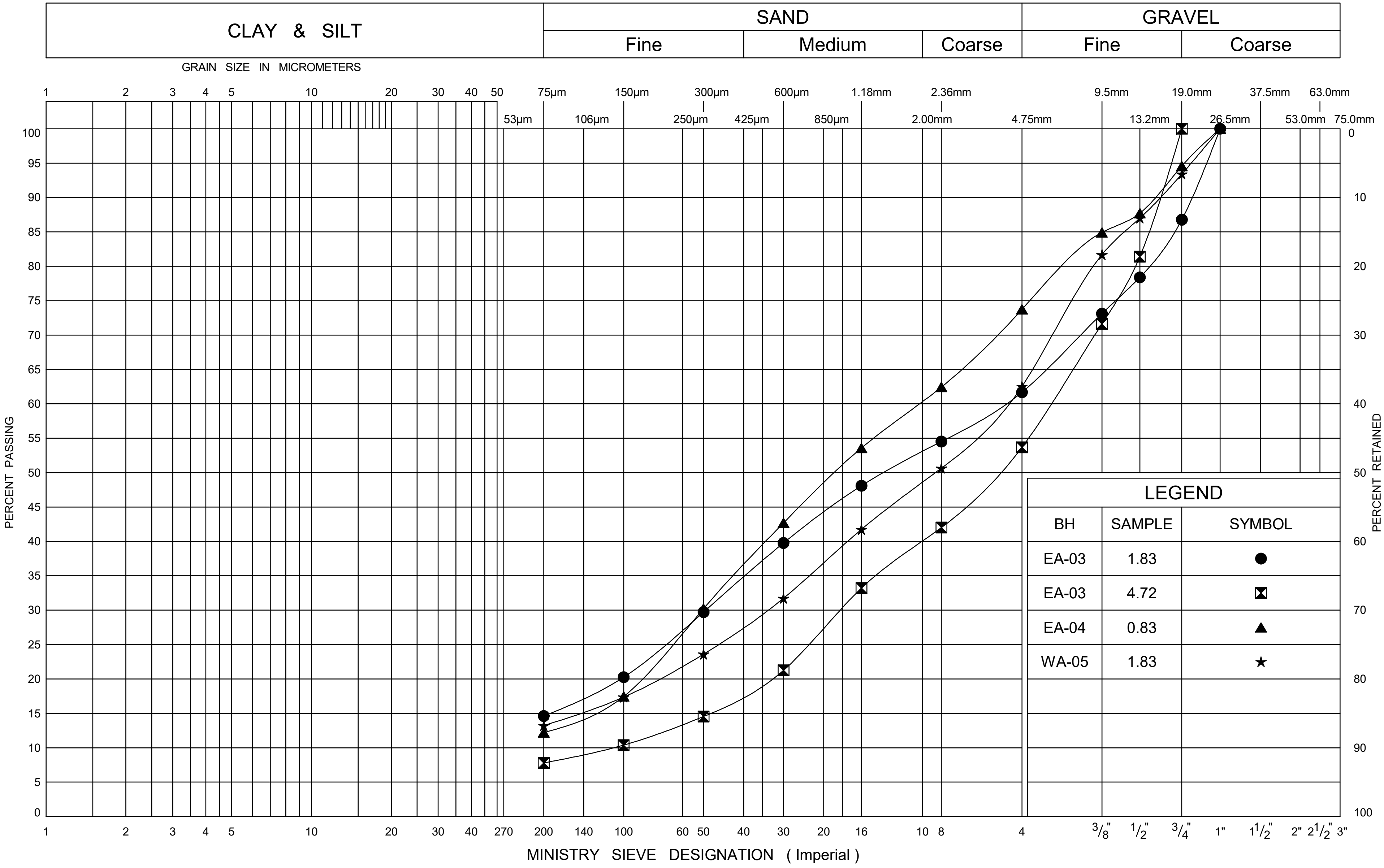
20
15-⊕-5
10
(%) STRAIN AT FAILURE

THURBER

Appendix C

Laboratory Test Results

78 12 M



ONTARIO MOT GRAIN SIZE 2 MTO-11373.GPJ ONTARIO MOT.GDT 11/21/18

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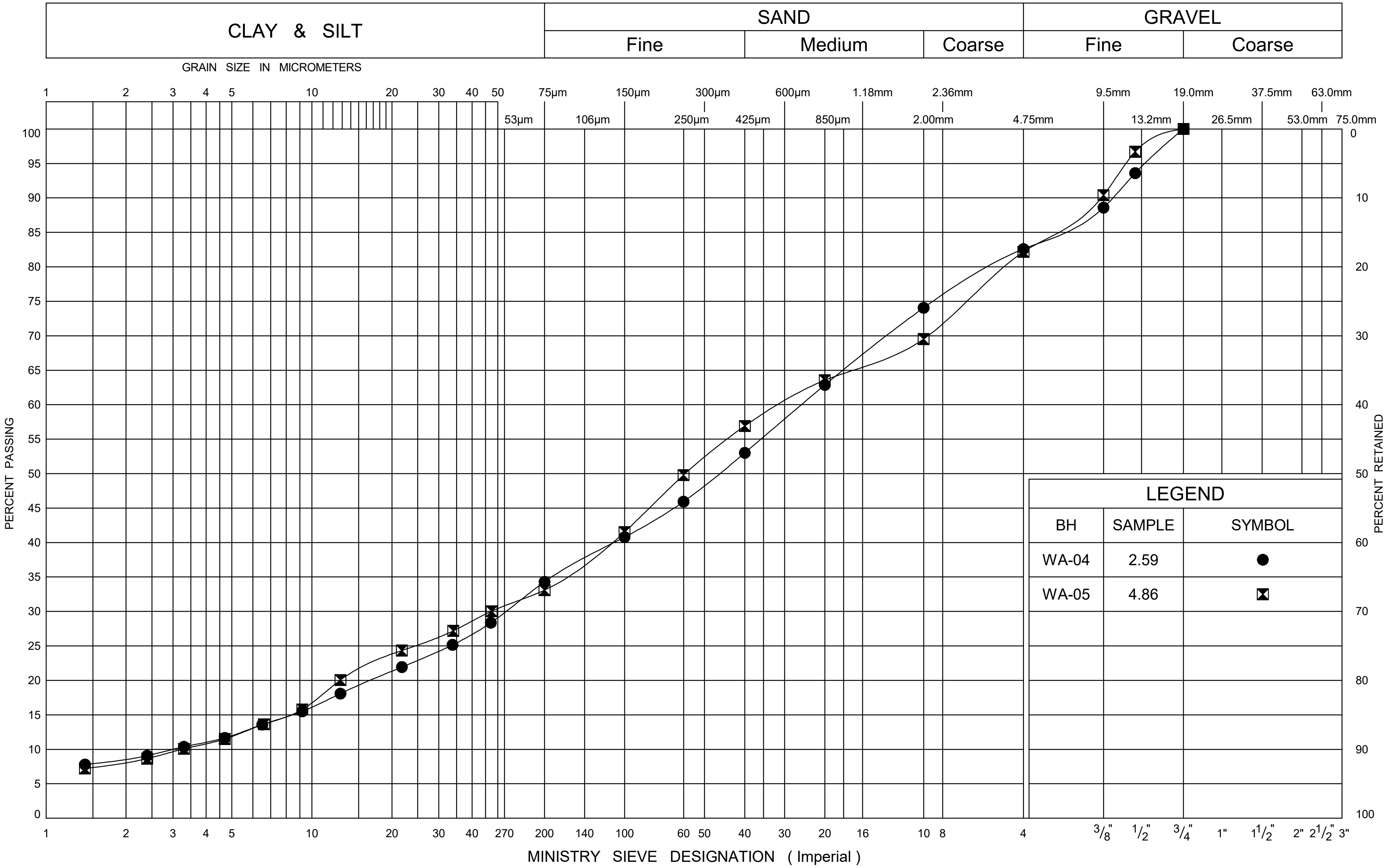
GRAIN SIZE DISTRIBUTION

SAND and GRAVEL FILL

FIG No C1

W P 3080-12-02/03

78 12 M



ONTARIO MOT GRAIN SIZE 2 MTO-11373.GPJ ONTARIO MOT.GDT 11/21/18

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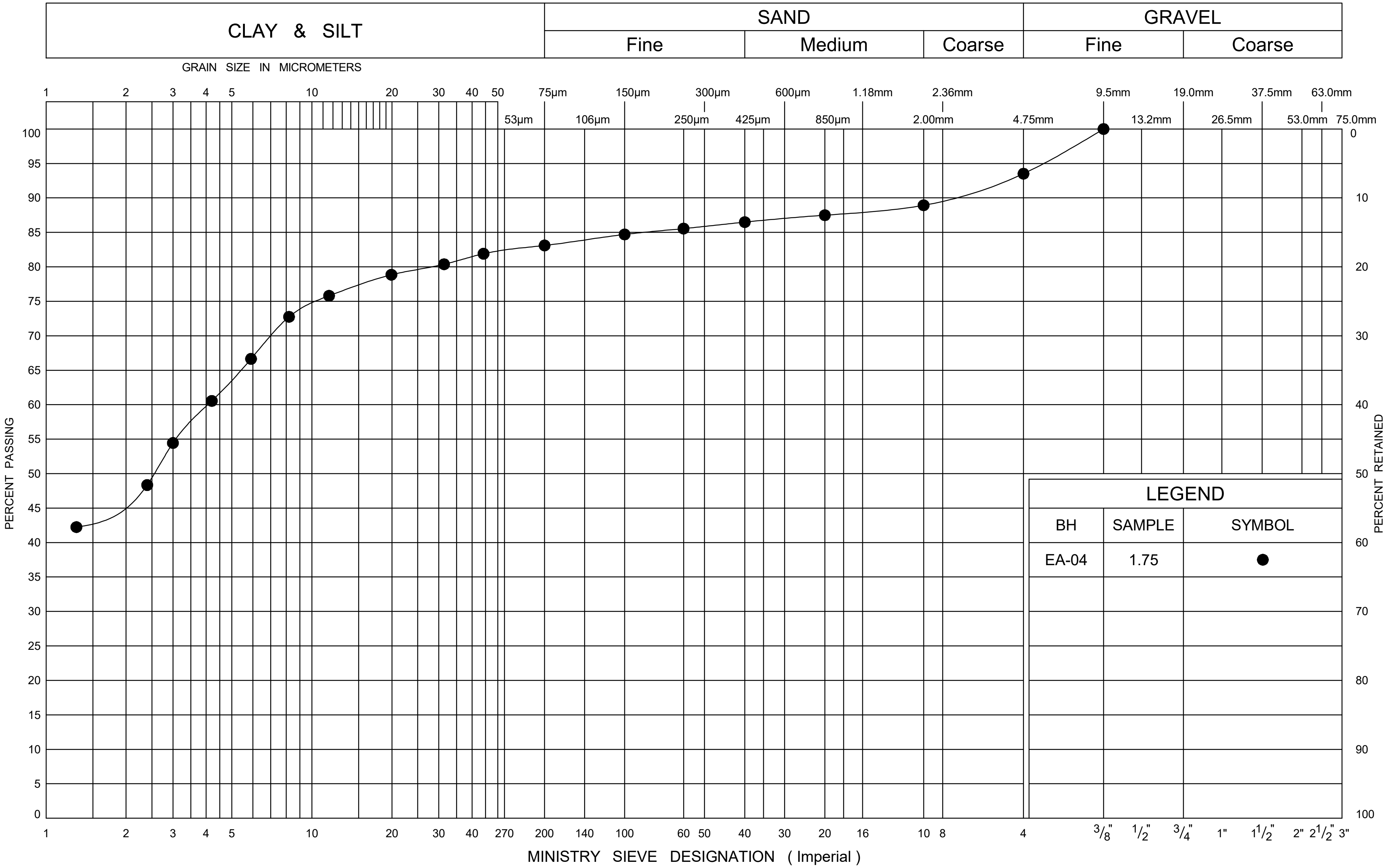
GRAIN SIZE DISTRIBUTION

Silty SAND FILL

FIG No C2

W P 3080-12-02/03

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
EA-04	1.75	●

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Ontario

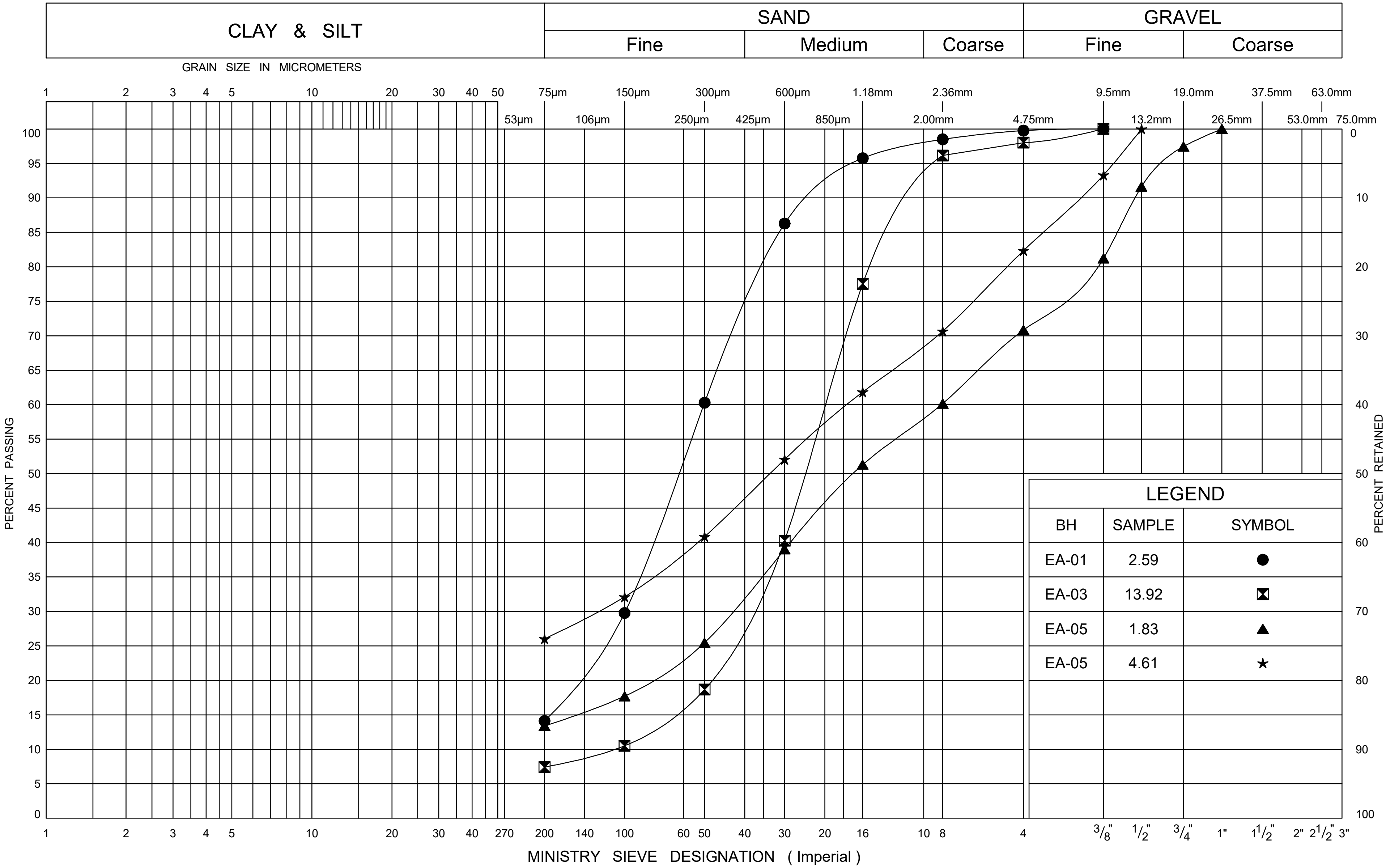
GRAIN SIZE DISTRIBUTION

CLAY TILL

FIG No C3

W P 3080-12-02/03

ONTARIO MOT GRAIN SIZE 2 MTO-11373.GPJ ONTARIO MOT.GDT 11/21/18



Ministry of
Transportation
Ontario

GRAIN SIZE DISTRIBUTION

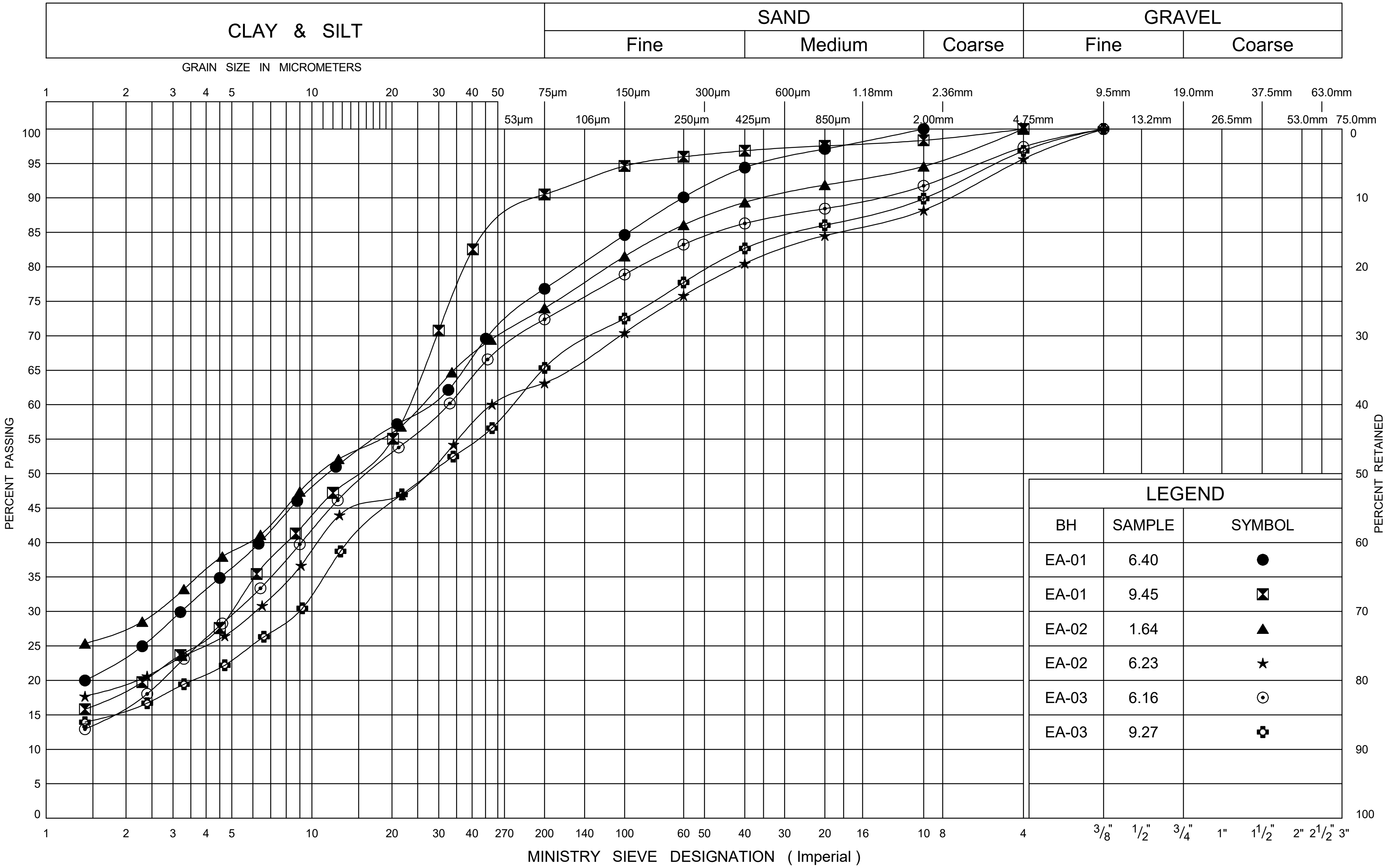
SAND

FIG No C4

W P 3080-12-02/03

ONTARIO MOT GRAIN SIZE 2 MTO-11373.GPJ ONTARIO MOT.GDT 11/21/18

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
EA-01	6.40	●
EA-01	9.45	☒
EA-02	1.64	▲
EA-02	6.23	★
EA-03	6.16	⊙
EA-03	9.27	✤

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GRAIN SIZE DISTRIBUTION

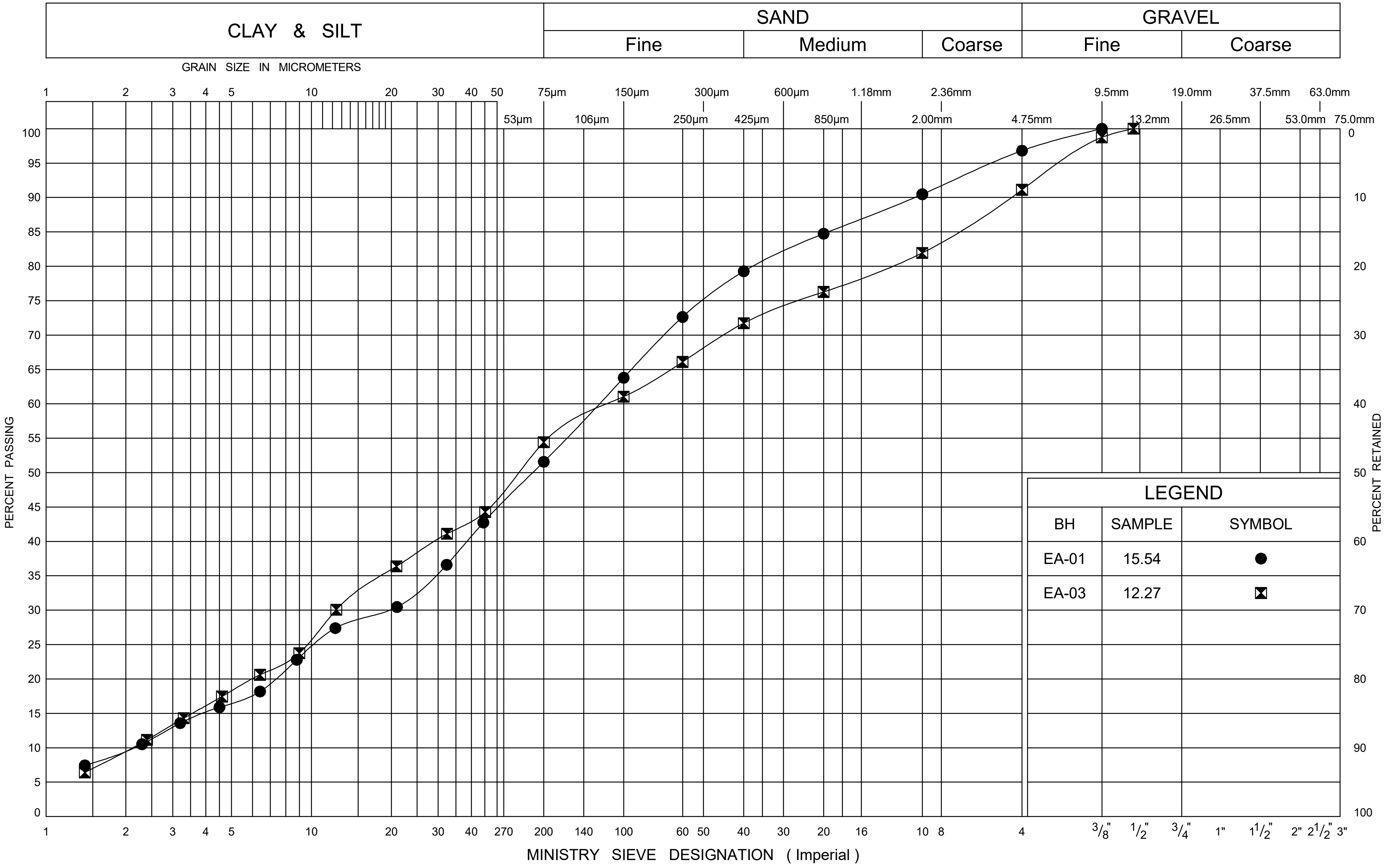
Silty CLAY to Clayey SILT TILL

FIG No C5

W P 3080-12-02/03

ONTARIO MOT GRAIN SIZE 2 MTO-11373.GPJ ONTARIO MOT.GDT 11/21/18

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
EA-01	15.54	●
EA-03	12.27	☒

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GRAIN SIZE DISTRIBUTION

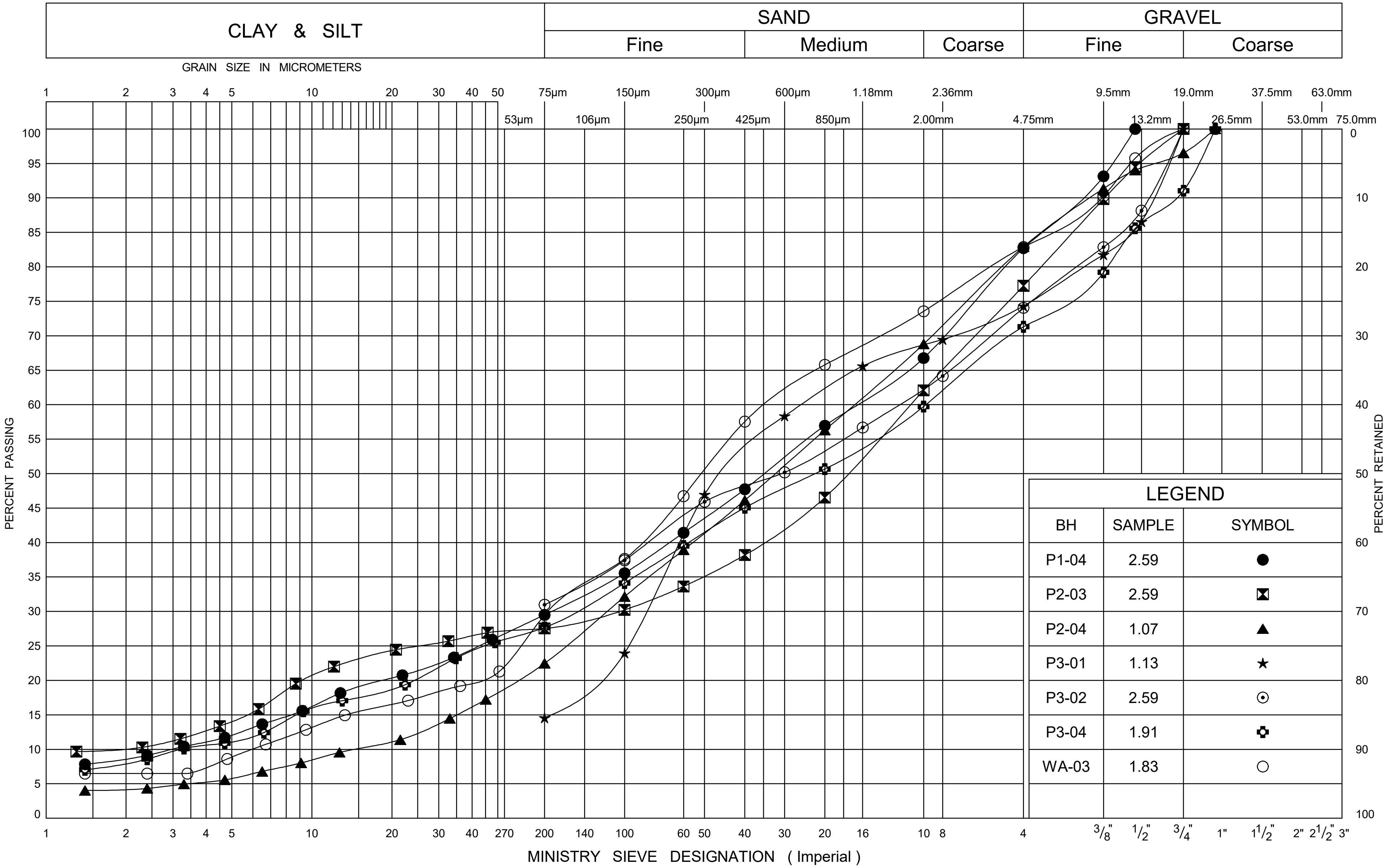
SILT and SAND TILL

FIG No C6

W P 3080-12-02/03

ONTARIO MOT GRAIN SIZE 2 MTO-11373.GPJ ONTARIO MOT.GDT 11/21/18

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
P1-04	2.59	●
P2-03	2.59	☒
P2-04	1.07	▲
P3-01	1.13	★
P3-02	2.59	⊙
P3-04	1.91	✚
WA-03	1.83	○

ONTARIO MOT GRAIN SIZE 2 MTO-11373.GPJ ONTARIO MOT.GDT 11/21/18

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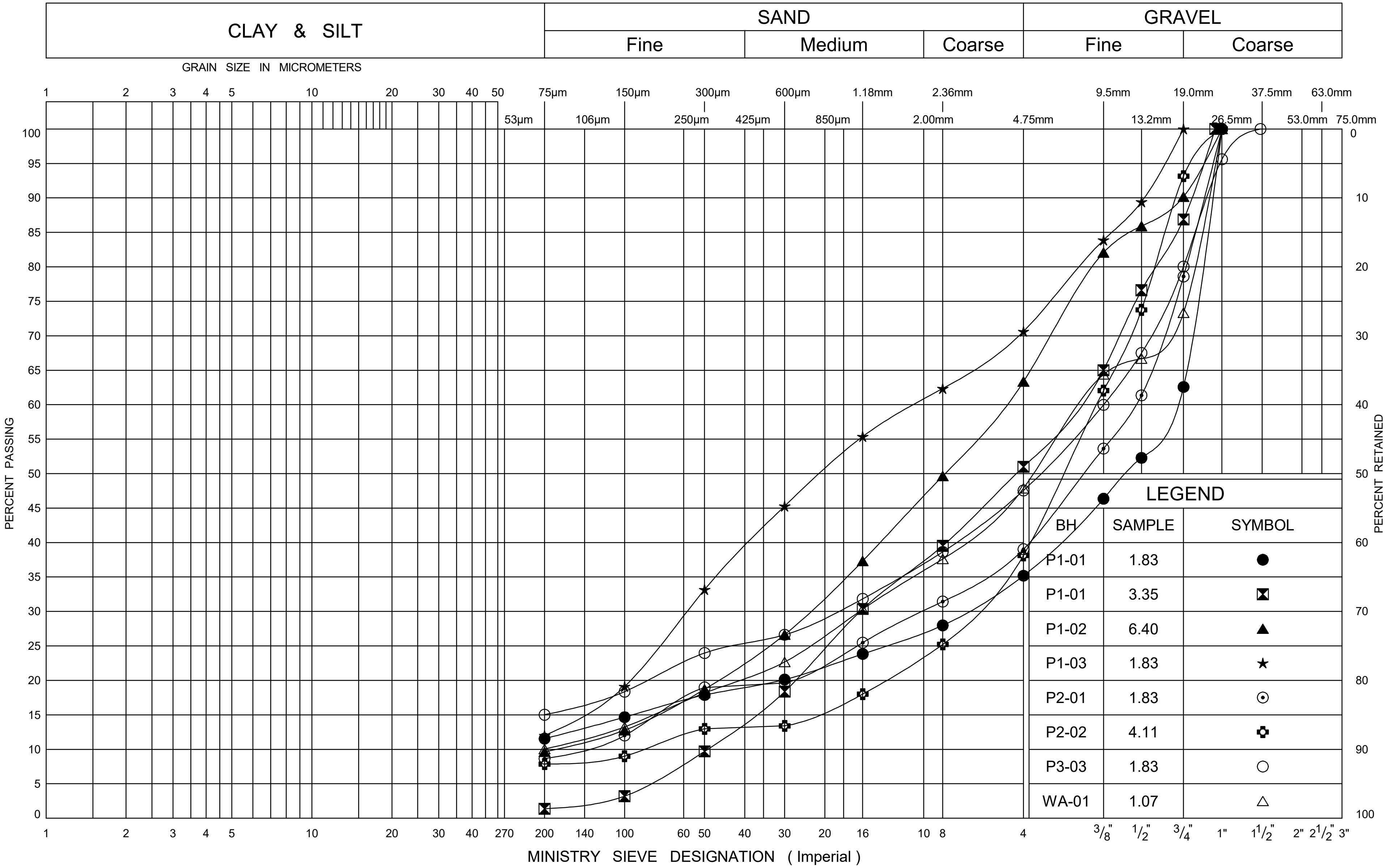
Transportation
Ontario

GRAIN SIZE DISTRIBUTION

Silty to Gravelly SAND

FIG No C7

W P 3080-12-02/03



LEGEND

BH	SAMPLE	SYMBOL
P1-01	1.83	●
P1-01	3.35	⧗
P1-02	6.40	▲
P1-03	1.83	★
P2-01	1.83	⊙
P2-02	4.11	✿
P3-03	1.83	○
WA-01	1.07	△

ONTARIO MOT GRAIN SIZE 2 MTO-11373.GPJ ONTARIO MOT.GDT 11/21/18

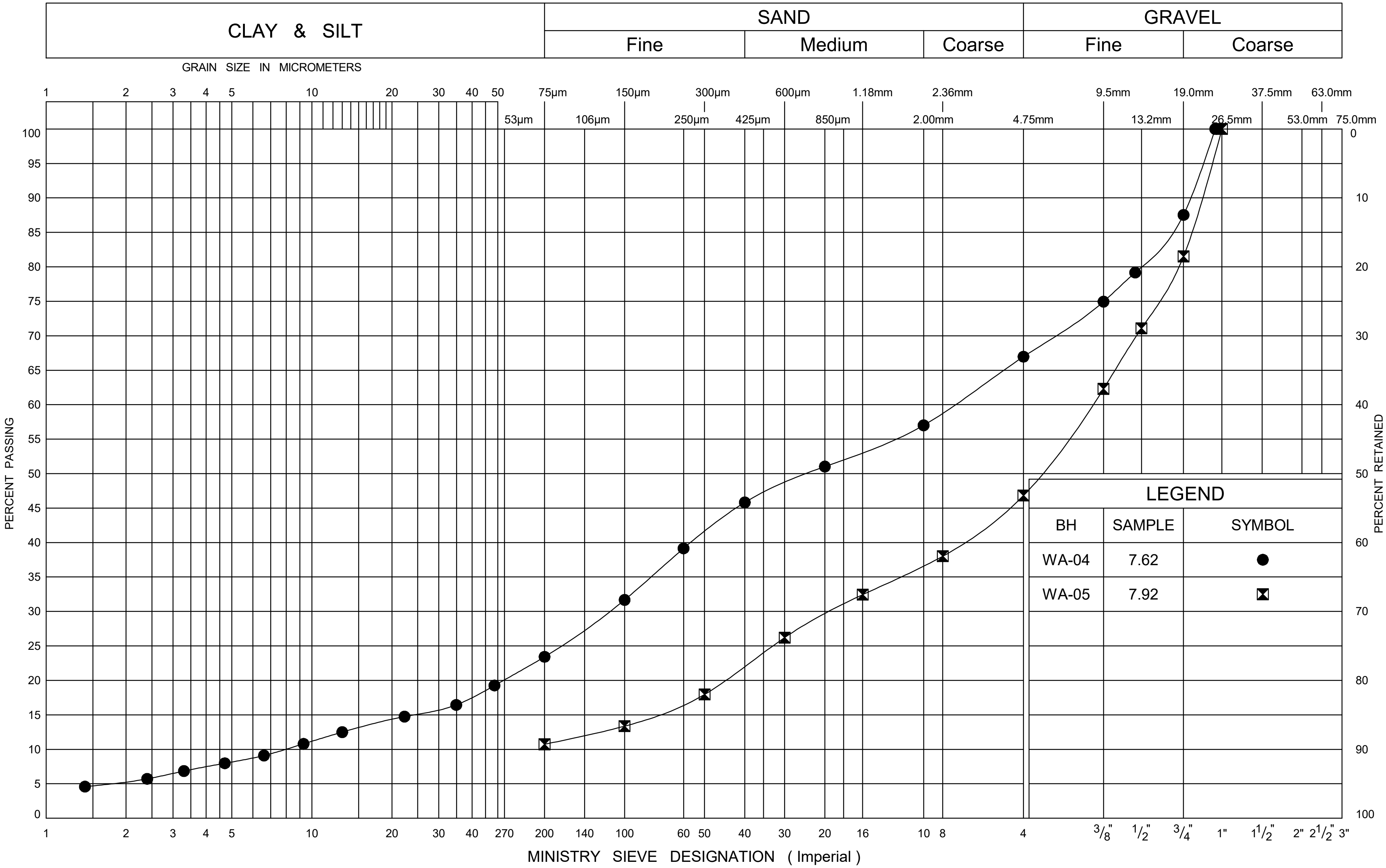
Ministry of
Transportation
Ontario

GRAIN SIZE DISTRIBUTION

SAND and GRAVEL

FIG No C8

W P 3080-12-02/03



LEGEND

BH	SAMPLE	SYMBOL
WA-04	7.62	●
WA-05	7.92	⊠

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Ontario

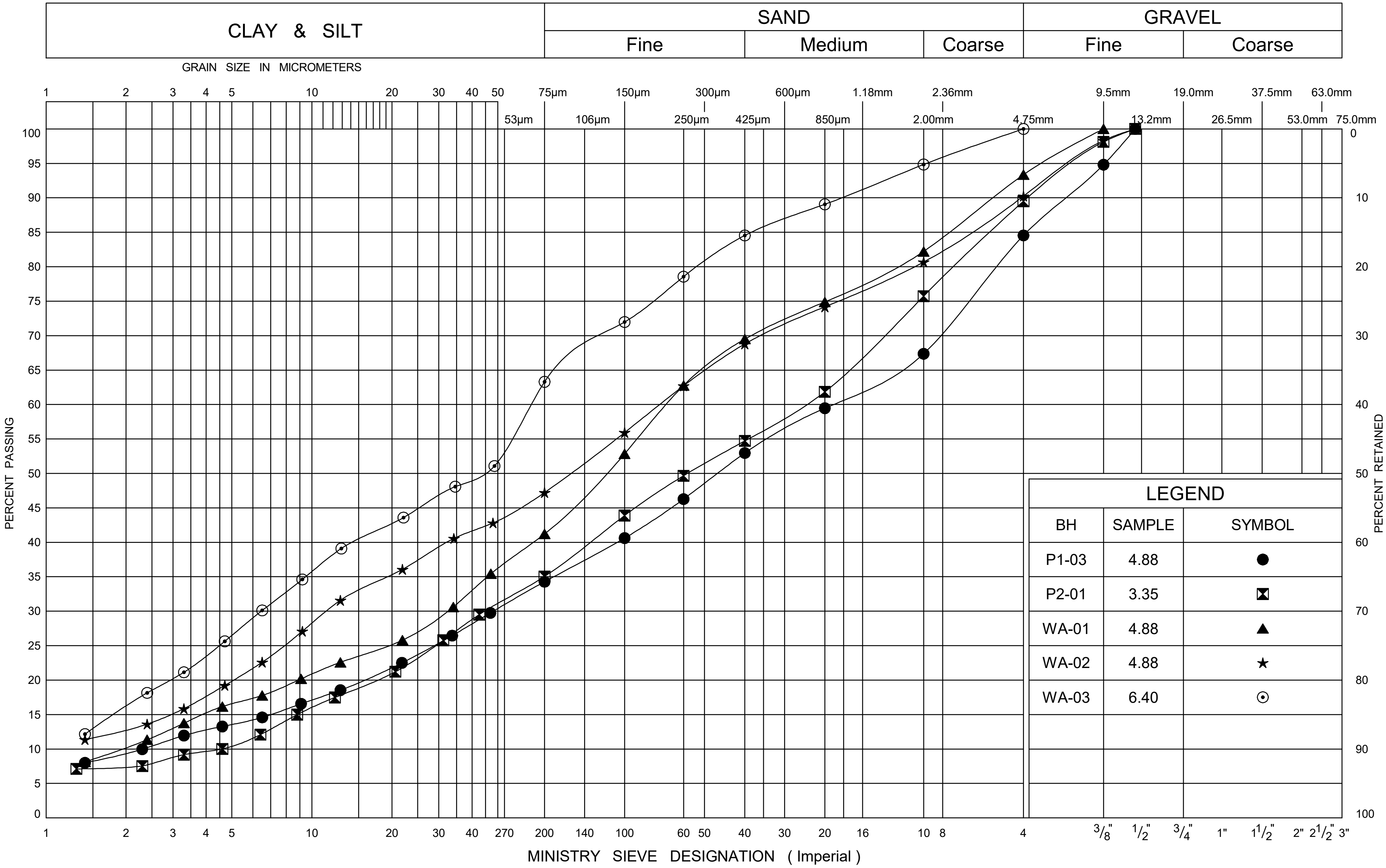
GRAIN SIZE DISTRIBUTION

SAND and GRAVEL

FIG No C9

W P 3080-12-02/03

ONTARIO MOT GRAIN SIZE 2 MTO-11373.GPJ ONTARIO MOT.GDT 11/21/18



BH	SAMPLE	SYMBOL
P1-03	4.88	●
P2-01	3.35	⊠
WA-01	4.88	▲
WA-02	4.88	★
WA-03	6.40	⊙

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GRAIN SIZE DISTRIBUTION

Silty SAND TILL

FIG No C10

W P 3080-12-02/03

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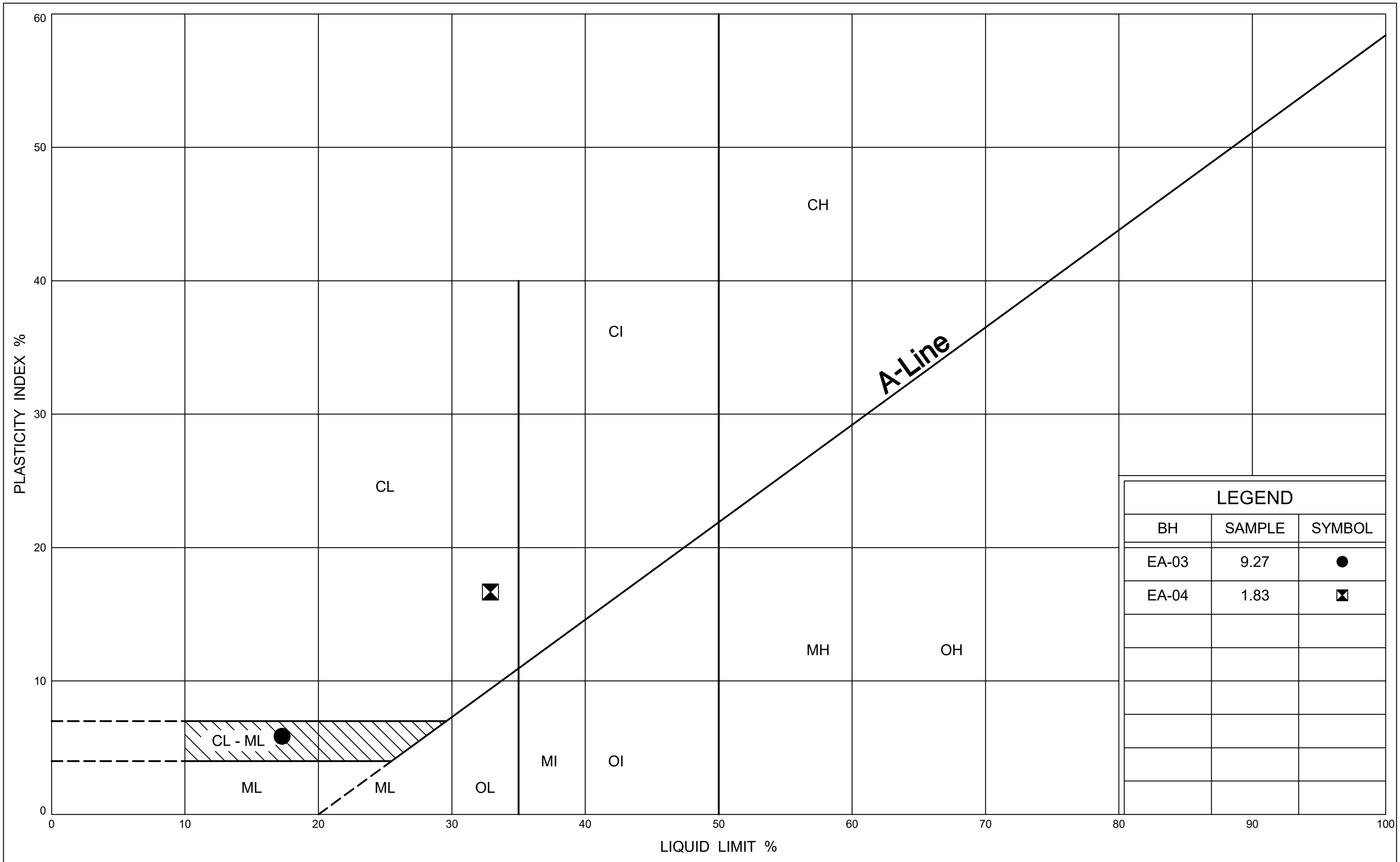

LEGEND

BH	SAMPLE	SYMBOL
EA-03	9.27	●
EA-04	1.83	⧗

Ministry of
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Ontario

PLASTICITY CHART

Silty CLAY to Clayey SILT TILL

FIG No C11

W P 3080-12-02/03

Uniaxial Compression Test

Client	Thurber Engineering Ltd.	**Project**	11373
Sample	WA-01-Run4	**Depth**	44’ 3” - 44’ 10”

Specimen parameters	
Diameter (mm) [a]	47.34
Length (mm) [a]	98.21
Bulk density ρ (g/cm^3)	2.319
UCS (MPa)	37.3
Lithology	Salina Formation: Limestone with several shaly layers and calcite veins
Failure description [b]	1

[a] Additional specimen measurement/details provides in accompanying summary spreadsheet.

[b] Failure description: [1] Inclined shear band failure;

Prior to testing

After testing

Remarks:

Performed by	BSAT	**Date**	2018-12-13

Uniaxial Compression Test

Client	Thurber Engineering Ltd.	**Project**	11373
Sample	WA-03-Run3	**Depth**	42’ 0” - 42’ 8”

Specimen parameters	
Diameter (mm) [a]	47.25
Length (mm) [a]	98.71
Bulk density ρ (g/cm^3)	2.299
UCS (MPa)	30.0
Lithology	Salina Formation - Limestone with high calcite content and shaly partings
Failure description [b]	2

[a] Additional specimen measurement/details provides in accompanying summary spreadsheet.
[b] Failure description: [2] Axial splitting failure;

Prior to testing

After testing

Remarks:

Performed by	BSAT	**Date**	2018-12-13

Uniaxial Compression Test

Client	Thurber Engineering Ltd.	**Project**	11373
Sample	P1-02-Run4	**Depth**	41’ 0” - 41’ 8”

Specimen parameters	
Diameter (mm) [a]	47.28
Length (mm) [a]	98.71
Bulk density ρ (g/cm^3)	2.706
UCS (MPa)	163.0
Lithology	Salina Formation - Limestone with few shaly partings
Failure description [b]	2

[a] Additional specimen measurement/details provides in accompanying summary spreadsheet.
[b] Failure description: [2] Axial splitting failure;

Prior to testing

After testing

Remarks:

Performed by	BSAT	**Date**	2018-12-13

Uniaxial Compression Test

Client	Thurber Engineering Ltd.	**Project**	11373
Sample	P1-04-Run5	**Depth**	40’ 5” - 41’ 0”

Specimen parameters

Diameter (mm) [a]	47.26
Length (mm) [a]	98.39
Bulk density ρ (g/cm^3)	2.704
UCS (MPa)	131.3
Lithology	Salina Formation - Limestone with few shaly partings
Failure description [b]	2

[a] Additional specimen measurement/details provides in accompanying summary spreadsheet.
[b] Failure description: [2] Axial splitting failure;

Prior to testing

After testing

Remarks:

Performed by	BSAT	**Date**	2018-12-13

Uniaxial Compression Test

Client	Thurber Engineering Ltd.	**Project**	11373
Sample	P2-01-Run4	**Depth**	32’ 9” - 33’ 3”

Specimen parameters	
Diameter (mm) [a]	47.43
Length (mm) [a]	97.52
Bulk density ρ (g/cm^3)	2.492
UCS (MPa)	95.1
Lithology	Salina Formation - Limestone with several voids throughout
Failure description [b]	1

[a] Additional specimen measurement/details provides in accompanying summary spreadsheet.
[b] Failure description: [1] Inclined shear band failure;

Prior to testing

After testing

Remarks:

Performed by	BSAT	**Date**	2018-12-13

Uniaxial Compression Test

Client	Thurber Engineering Ltd.	**Project**	11373
Sample	P2-03-Run7	**Depth**	43’ 3” - 44’ 0”

Specimen parameters	
Diameter (mm) [a]	47.40
Length (mm) [a]	98.42
Bulk density ρ (g/cm^3)	2.704
UCS (MPa)	61.2
Lithology	Salina Formation - Limestone with few shaly partings
Failure description [b]	2

[a] Additional specimen measurement/details provides in accompanying summary spreadsheet.
[b] Failure description: [2] Axial splitting failure;

Prior to testing

After testing

Remarks:

Performed by	BSAT	**Date**	2018-12-13

Uniaxial Compression Test

Client	Thurber Engineering Ltd.	**Project**	11373
Sample	P3-02-Run6	**Depth**	39’ 10” - 40’ 5”

Specimen parameters	
Diameter (mm) [a]	47.23
Length (mm) [a]	98.18
Bulk density ρ (g/cm^3)	2.701
UCS (MPa)	115.0
Lithology	Salina Formation - Limestone with several shaly layers and partings
Failure description [b]	2

[a] Additional specimen measurement/details provides in accompanying summary spreadsheet.

[b] Failure description: [2] Axial splitting failure;

Prior to testing

After testing

Remarks:

Performed by	BSAT	**Date**	2018-12-13

Uniaxial Compression Test

Client	Thurber Engineering Ltd.	**Project**	11373
Sample	EA-03-Run6	**Depth**	79’ 7” - 80’ 0”

Specimen parameters

Diameter (mm) [a]	47.51
Length (mm) [a]	98.13
Bulk density ρ (g/cm^3)	2.709
UCS (MPa)	128.3
Lithology	Salina Formation - Limestone with few shaly partings
Failure description [b]	2

[a] Additional specimen measurement/details provides in accompanying summary spreadsheet.

[b] Failure description: ^{2} Axial splitting failure;

Prior to testing

After testing

Remarks:

Performed by	BSAT	**Date**	2018-12-13

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11373	**Date Drilled:**	14-Mar-17
Client:	WSP	**Date Tested:**	16-Mar-17
Project Name:	401 Grand River	**Tester:**	KF
Core Size:	NQ **BH No :** WA-01	**Reviewed by:**	KAF

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	9.7	D	16.4	47.3	67.2	6.8	163.0	Salina	Very Strong
2	2	11.3	D	15.0	47.3	62.2	6.2	149.1	Salina	Very Strong
3	3	11.9	D	26.2	47.3	69.3	10.8	259.9	Salina	Extremely Strong
4	4	13.4	D	8.5	47.2	75.8	3.5	84.2	Salina	Strong
5	4	14.1	D	5.7	47.0	124.8	2.4	57.0	Salina	Strong
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* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

* Correlation factor to obtain UCS values is 24.

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11373	**Date Drilled:**	14-Mar-17
Client:	WSP	**Date Tested:**	16-Mar-17
Project Name:	401 Grand River	**Tester:**	KF
Core Size:	NQ **BH No :** WA-02	**Reviewed by:**	KAF

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	8.9	D	4.5	47.3	67.8	1.9	45.0	Salina	Medium Strong
2	2	10.1	D	8.3	47.2	66.5	3.4	82.4	Salina	Strong
3	2	10.4	D	7.2	47.2	59.3	3.0	72.1	Salina	Strong
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THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11373	**Date Drilled:**	14-Mar-17
Client:	WSP	**Date Tested:**	16-Mar-17
Project Name:	401 Grand River	**Tester:**	KF
Core Size:	NQ **BH No :** WA-03	**Reviewed by:**	KAF

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	3	12.8	D	6.5	46.9	101.5	2.7	65.5	Salina	Strong
2	3	12.9	D	7.3	46.4	247.2	3.1	74.4	Salina	Strong
3	3	13.1	D	3.2	46.9	313.5	1.3	31.8	Salina	Medium Strong
4	4	13.6	D	24.6	47.2	101.5	10.2	244.9	Salina	Very Strong
5	4	14.0	D	8.6	47.1	145.8	3.6	85.4	Salina	Strong
6	5	15.1	D	2.0	47.4	77.2	0.8	19.4	Salina	Weak
7	5	16.3	D	5.0	47.4	55.5	2.0	49.1	Salina	Medium Strong
8	6	16.7	D	2.2	47.3	92.4	0.9	22.0	Salina	Weak
9	6	17.3	D	7.4	47.4	114.9	3.1	73.3	Salina	Strong
10	6	17.8	D	7.3	47.3	111.7	3.0	72.5	Salina	Strong
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Last Modified: September 14, 2016

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11373	**Date Drilled:**	14-Mar-17
Client:	WSP	**Date Tested:**	16-Mar-17
Project Name:	401 Grand River	**Tester:**	KF
Core Size:	NQ **BH No :** P1-01	**Reviewed by:**	KAF

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	8.4	D	11.5	47.3	58.3	4.8	114.1	Salina	Very Strong
2	1	8.8	D	6.8	47.2	101.5	2.8	67.4	Salina	Strong
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THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11373	**Date Drilled:**	14-Mar-17
Client:	WSP	**Date Tested:**	16-Mar-17
Project Name:	401 Grand River	**Tester:**	KF
Core Size:	NQ **BH No :** P1-02	**Reviewed by:**	KAF

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	7.4	D	5.7	46.6	48.8	2.4	57.6	Salina	Strong
2	1	7.8	A	24.9	47.1	66.1	6.6	158.7	Salina	Very Strong
3	3	10.6	D	22.4	47.3	113.9	9.3	222.3	Salina	Very Strong
4	4	12.1	D	10.0	47.1	62.6	4.2	99.6	Salina	Strong
5	4	12.8	D	18.9	47.2	72.1	7.8	187.8	Salina	Very Strong
6	5	13.7	D	12.3	47.0	147.3	5.2	123.6	Salina	Very Strong
7	5	14.3	D	2.4	47.2	130.5	1.0	23.5	Salina	Weak
8	6	15.0	D	4.5	47.1	91.8	1.9	44.5	Salina	Medium Strong
9	6	16.1	D	9.1	47.3	90.5	3.8	90.0	Salina	Strong
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THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11373	**Date Drilled:**	14-Mar-17
Client:	WSP	**Date Tested:**	16-Mar-17
Project Name:	401 Grand River	**Tester:**	KF
Core Size:	NQ **BH No :** P1-03	**Reviewed by:**	KAF

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	7.1	D	6.3	47.0	41.1	2.6	62.6	Salina	Strong
2	2	8.3	D	7.1	47.1	63.5	3.0	70.9	Salina	Strong
3	2	9.1	D	2.6	47.1	110.8	1.1	26.1	Salina	Medium Strong
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THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11373		**Date Drilled:**	14-Mar-17
Client:	WSP		**Date Tested:**	16-Mar-17
Project Name:	401 Grand River		**Tester:**	KF
Core Size:	NQ	**BH No :** P1-04	**Reviewed by:**	KAF

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	6.3	D	9.6	47.3	89.6	3.9	94.7	Salina	Strong
2	2	7.6	D	4.4	47.0	59.8	1.8	44.1	Salina	Medium Strong
3	5	12.2	D	25.9	47.2	83.9	10.7	257.9	Salina	Extremely Strong
4	5	13.1	D	19.7	47.1	101.1	8.2	196.6	Salina	Very Strong
5	5	13.4	D	24.2	47.3	85.5	10.0	239.6	Salina	Very Strong
6	6	13.8	D	7.2	47.1	94.4	3.0	72.1	Salina	Strong
7	6	14.1	D	9.7	47.2	94.9	4.0	96.4	Salina	Strong
8	6	14.5	D	4.3	47.2	84.7	1.8	42.3	Salina	Medium Strong
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Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

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THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11373	**Date Drilled:**	14-Mar-17
Client:	WSP	**Date Tested:**	16-Mar-17
Project Name:	401 Grand River	**Tester:**	KF
Core Size:	NQ **BH No :** P2-01	**Reviewed by:**	KAF

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	5.5	D	7.4	47.1	52.6	3.1	74.1	Salina	Strong
2	2	8.3	D	3.3	47.1	61.4	1.4	33.4	Salina	Medium Strong
3	3	9.5	D	12.2	47.1	59.2	5.1	122.3	Salina	Very Strong
4	3	9.8	D	23.4	47.2	51.4	9.7	233.1	Salina	Very Strong
5	4	10.2	D	17.3	47.6	60.5	7.1	169.7	Salina	Very Strong
6	4	10.5	D	4.4	47.4	75.8	1.8	43.3	Salina	Medium Strong
7	4	10.7	D	1.3	47.4	61.3	0.6	13.3	Salina	Weak
8	4	11.1	D	7.6	47.2	51.8	3.2	75.9	Salina	Strong
9	4	11.4	A	7.8	47.2	49.8	2.6	61.9	Salina	Strong
10	5	11.6	D	5.6	47.2	52.3	2.3	56.2	Salina	Strong
11	5	11.9	D	11.4	47.1	51.8	4.7	113.5	Salina	Very Strong
12	5	12.2	D	22.9	47.2	65.2	9.5	227.9	Salina	Very Strong
13	5	12.6	D	3.0	47.2	58.4	1.2	29.7	Salina	Medium Strong
14	5	12.8	D	12.3	47.1	54.0	5.1	122.7	Salina	Very Strong
15	6	13.1	D	14.2	47.0	52.1	5.9	142.5	Salina	Very Strong
16	6	13.4	D	17.1	47.2	60.6	7.1	170.3	Salina	Very Strong
17	6	13.7	D	16.2	47.2	62.3	6.7	161.0	Salina	Very Strong
18	6	14.1	D	4.0	47.2	81.9	1.7	39.8	Salina	Medium Strong
19	6	14.4	D	17.7	47.2	60.9	7.3	175.7	Salina	Very Strong
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Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

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* Correlation factor to obtain UCS values is 24.

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11373	**Date Drilled:**	14-Mar-17
Client:	WSP	**Date Tested:**	16-Mar-17
Project Name:	401 Grand River	**Tester:**	KF
Core Size:	NQ **BH No :** P2-02	**Reviewed by:**	KAF

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	5.3	D	3.9	47.1	60.3	1.6	39.4	Salina	Medium Strong
2	2	5.6	D	3.7	47.0	72.4	1.6	37.5	Salina	Medium Strong
3	2	5.9	D	3.2	47.2	54.1	1.3	31.7	Salina	Medium Strong
4	3	7.1	D	11.0	47.2	57.6	4.6	109.3	Salina	Very Strong
5	3	7.5	D	4.9	47.1	56.9	2.0	48.8	Salina	Medium Strong
6	3	7.7	A	5.8	47.5	48.8	1.9	46.2	Salina	Medium Strong
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THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No: 11373

Client: WSP

Project Name: 401 Grand River

Core Size: NQ **BH No :** P2-03

Date Drilled: 14-Mar-17

Date Tested: 16-Mar-17

Tester: KF

Reviewed by: KAF

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	4.5	D	1.7	47.5	42.8	0.7	16.8	Salina	Weak
2	2	6.0	D	12.5	47.2	77.8	5.2	124.0	Salina	Very Strong
3	3	7.1	D	2.7	46.7	52.1	1.1	27.6	Salina	Medium Strong
4	3	7.3	D	7.2	47.2	62.1	3.0	72.0	Salina	Strong
5	3	7.7	D	14.4	47.3	74.5	6.0	143.2	Salina	Very Strong
6	4	8.6	D	17.5	47.3	66.2	7.3	174.1	Salina	Very Strong
7	4	8.8	D	24.8	47.3	57.2	10.2	246.0	Salina	Very Strong
8	4	9.1	D	0.8	47.1	72.7	0.3	8.0	Salina	Weak
9	5	9.8	D	5.3	47.1	58.8	2.2	53.0	Salina	Strong
10	5	10.8	D	16.5	47.2	70.9	6.9	164.5	Salina	Very Strong
11	5	11.0	A	18.3	47.4	54.6	5.6	134.2	Salina	Very Strong
12	6	11.3	A	3.1	47.2	49.0	1.0	24.8	Salina	Weak
13	6	11.6	A	1.7	47.3	48.8	0.6	13.7	Salina	Weak
14	6	11.9	D	2.9	47.5	75.1	1.2	28.7	Salina	Medium Strong
15	6	12.2	A	3.9	47.3	48.4	1.3	31.7	Salina	Medium Strong
16	7	12.7	D	6.1	47.3	50.1	2.5	60.5	Salina	Strong
17	7	13.1	D	6.7	47.3	53.0	2.8	66.3	Salina	Strong
18	7	13.4	D	8.0	47.3	57.5	3.3	79.4	Salina	Strong
19	7	13.8	D	6.5	47.4	50.4	2.7	64.5	Salina	Strong
20	7	14.1	D	9.4	47.2	51.4	3.9	93.2	Salina	Strong
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Last Modified: September 14, 2016

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11373	**Date Drilled:**	14-Mar-17
Client:	WSP	**Date Tested:**	16-Mar-17
Project Name:	401 Grand River	**Tester:**	KF
Core Size:	NQ **BH No :** P2-04	**Reviewed by:**	KAF

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	2.0	D	2.6	47.8	71.9	1.0	25.2	Salina	Medium Strong
2	1	2.2	D	8.4	47.4	59.3	3.4	82.7	Salina	Strong
3	2	2.7	D	6.8	47.2	78.1	2.8	67.4	Salina	Strong
4	3	4.1	D	5.7	47.2	74.8	2.4	56.5	Salina	Strong
5	3	4.5	D	7.8	47.2	73.6	3.2	78.0	Salina	Strong
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THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No: 11373
Date Drilled: 14-Mar-17
Client: WSP
Date Tested: 16-Mar-17
Project Name: 401 Grand River
Tester: KF
Core Size: NQ **BH No :** P3-01
Reviewed by: KAF

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	4.4	D	4.9	47.0	75.1	2.0	48.7	Salina	Medium Strong
2	2	5.6	D	14.6	46.9	107.5	6.1	146.7	Salina	Very Strong
3	2	5.8	D	3.4	46.9	147.0	1.4	34.0	Salina	Medium Strong
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THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11373	**Date Drilled:**	14-Mar-17
Client:	WSP	**Date Tested:**	16-Mar-17
Project Name:	401 Grand River	**Tester:**	KF
Core Size:	NQ **BH No :** P3-02	**Reviewed by:**	KAF

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	4.6	D	6.1	46.8	84.2	2.6	61.7	Salina	Strong
2	2	6.1	D	7.8	46.9	94.7	3.3	78.4	Salina	Strong
3	3	8.2	D	3.6	46.7	65.1	1.5	36.2	Salina	Medium Strong
4	4	9.2	D	18.7	46.8	72.4	7.8	188.4	Salina	Very Strong
5	5	10.1	D	24.6	47.0	74.0	10.3	246.0	Salina	Very Strong
6	5	11.0	D	5.6	46.9	133.5	2.3	55.9	Salina	Strong
7	5	11.4	D	11.1	46.8	110.5	4.7	112.1	Salina	Very Strong
8	6	12.0	D	5.1	46.9	106.3	2.1	51.3	Salina	Strong
9	6	12.5	D	6.4	46.9	102.5	2.7	64.1	Salina	Strong
10	6	13.0	D	19.4	46.9	131.4	8.1	195.4	Salina	Very Strong
11	7	13.2	D	15.5	46.9	82.1	6.5	155.8	Salina	Very Strong
12	7	14.2	D	17.1	46.7	101.6	7.2	173.4	Salina	Very Strong
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THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11373	**Date Drilled:**	14-Mar-17
Client:	WSP	**Date Tested:**	16-Mar-17
Project Name:	401 Grand River	**Tester:**	KF
Core Size:	NQ **BH No :** P3-03	**Reviewed by:**	KAF

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	2	4.8	D	0.5	46.7	74.0	0.2	5.3	Salina	Weak
2	2	5.2	D	17.0	46.7	63.6	7.2	172.0	Salina	Very Strong
3	4	7.6	D	12.9	46.5	86.1	5.5	131.6	Salina	Very Strong
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THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11373		**Date Drilled:**	14-Mar-17
Client:	WSP		**Date Tested:**	16-Mar-17
Project Name:	401 Grand River		**Tester:**	KF
Core Size:	NQ	**BH No :** P3-04	**Reviewed by:**	KAF

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	2	5.2	D	0.9	46.7	61.7	0.4	8.7	Salina	Weak
2	3	7.0	D	1.5	46.9	72.8	0.6	14.7	Salina	Weak
3	4	8.1	D	13.5	47.0	66.2	5.6	135.1	Salina	Very Strong
4	4	8.2	D	8.8	47.4	59.1	3.6	86.7	Salina	Strong
5	4	8.6	D	12.2	47.1	30.7	5.1	121.5	Salina	Very Strong
6	5	10.0	D	8.3	47.1	73.6	3.5	83.1	Salina	Strong
7	5	10.4	D	10.5	47.1	68.0	4.4	105.3	Salina	Very Strong
8	5	10.9	D	10.7	47.1	56.5	4.4	106.5	Salina	Very Strong
9	6	11.7	D	14.7	47.3	86.2	6.1	146.4	Salina	Very Strong
10	6	12.5	D	14.1	47.1	60.4	5.9	141.0	Salina	Very Strong
11	7	12.8	D	10.8	47.2	64.3	4.5	107.3	Salina	Very Strong
12	7	13.8	D	17.5	47.2	57.9	7.3	174.6	Salina	Very Strong
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THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No: 11373 **Date Drilled:** 14-Mar-17
Client: WSP **Date Tested:** 16-Mar-17
Project Name: 401 Grand River **Tester:** KF
Core Size: NQ **BH No :** EA-01 **Reviewed by:** KAF

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	18.6	A	12.5	47.2	40.6	4.8	116.3	Salina	Very Strong
2	2	18.8	D	5.8	47.2	110.2	2.4	57.4	Salina	Strong
3	2	19.0	A	14.3	47.2	55.8	4.3	103.8	Salina	Very Strong
4	2	20.1	D	11.4	47.2	78.4	4.7	113.8	Salina	Very Strong
5	3	20.3	A	18.7	47.2	47.5	6.4	153.6	Salina	Very Strong
6	3	20.7	A	4.7	47.2	40.2	1.8	43.6	Salina	Medium Strong
7	4	21.9	A	6.8	47.2	48.1	2.3	55.2	Salina	Strong
8	4	22.3	A	5.4	47.2	46.2	1.9	45.5	Salina	Medium Strong
9	4	22.4	D	19.6	47.1	66.8	8.2	195.6	Salina	Very Strong
10	4	22.9	A	9.3	47.2	54.5	2.9	68.6	Salina	Strong
11	5	23.3	D	5.4	47.1	112.6	2.3	54.2	Salina	Strong
12	5	23.7	A	5.9	47.2	50.4	1.9	46.5	Salina	Medium Strong
13	5	23.9	D	18.3	47.2	94.5	7.6	182.1	Salina	Very Strong
14	5	24.2	A	4.8	47.2	57.5	1.4	33.9	Salina	Medium Strong
15	5	24.6	D	17.3	47.2	72.8	7.2	172.8	Salina	Very Strong
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THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11373	**Date Drilled:**	14-Mar-17
Client:	WSP	**Date Tested:**	16-Mar-17
Project Name:	401 Grand River	**Tester:**	KF
Core Size:	NQ **BH No :** EA-02	**Reviewed by:**	KAF

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	10.3	D	10.8	46.7	153.4	4.6	109.7	Salina	Very Strong
2	1	11.3	D	1.9	46.7	73.1	0.8	19.0	Salina	Weak
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THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No: 11373
Client: WSP
Project Name: 401 Grand River
Core Size: NQ **BH No :** EA-03

Date Drilled: 14-Mar-17
Date Tested: 16-Mar-17
Tester: KF
Reviewed by: KAF

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	2	20.1	A	25.0	47.0	56.0	7.5	181.2	Salina	Very Strong
2	5	21.8	D	9.5	47.3	64.5	3.9	93.8	Salina	Strong
3	5	22.1	D	25.9	47.1	85.9	10.8	259.0	Salina	Extremely Strong
4	5	22.9	A	19.3	47.0	38.3	7.8	187.9	Salina	Very Strong
5	6	23.4	A	2.7	47.0	39.1	1.1	25.5	Salina	Medium Strong
6	6	23.5	D	9.5	47.0	131.0	4.0	95.0	Salina	Strong
7	6	23.7	D	11.5	47.0	102.3	4.8	115.6	Salina	Very Strong
8	6	24.1	D	12.6	47.0	130.7	5.3	126.6	Salina	Very Strong
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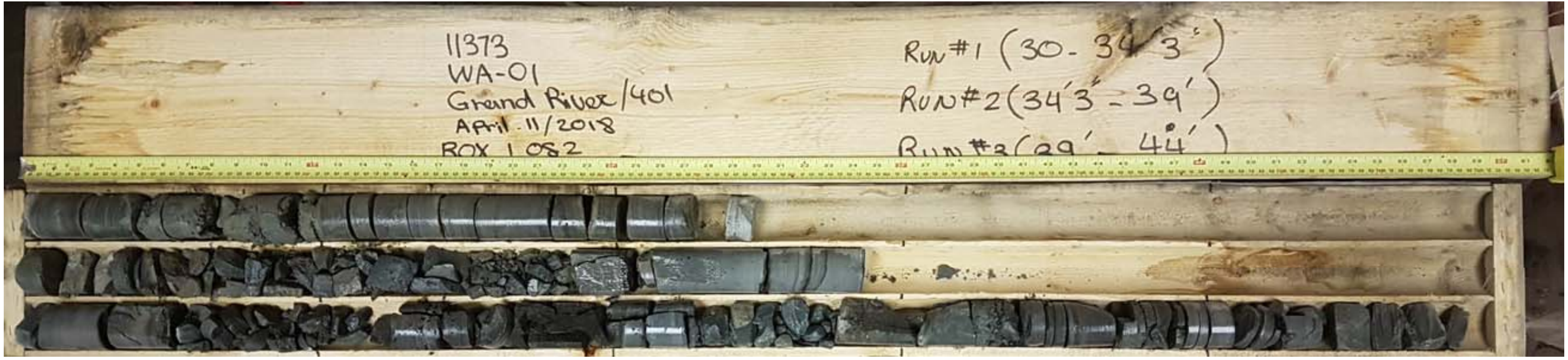
Appendix D

Photographs of Bedrock Core

Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole WA-01 – Runs 1, 2 and 3



Borehole WA-01 – Run 4

THURBER

Grand River Bridge Replacement

Photographs of Bedrock Core

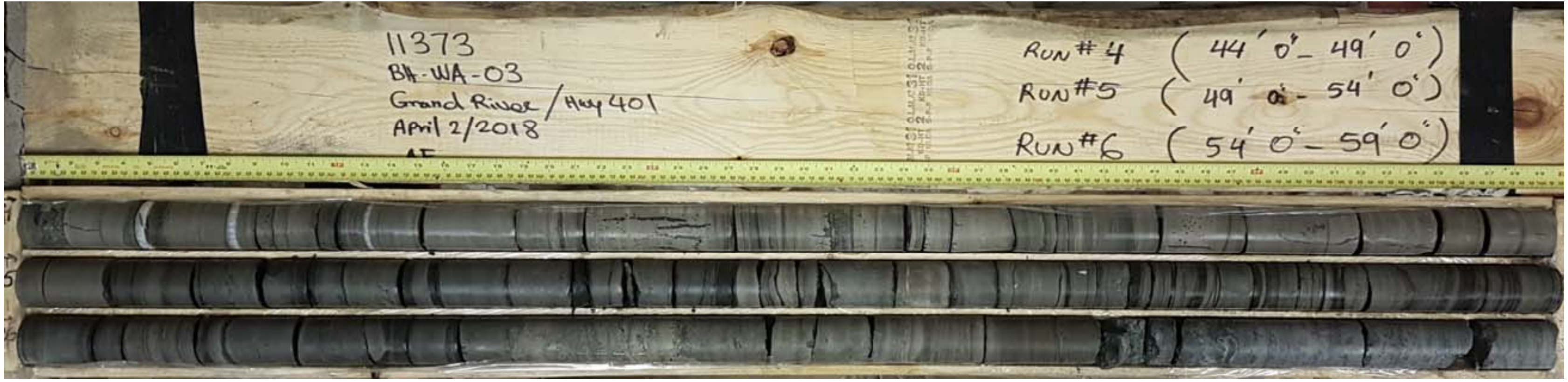
Borehole WA-02 – Runs 1 and 2

Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole WA-03 – Runs 1, 2 and 3

Borehole WA-03 – Runs 4, 5 and 6



Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole P1-01 – Runs 1 and 2

Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole P1-02 – Runs 1, 2 and 3

Borehole P1-02 – Runs 4, 5 and 6

Borehole P1-03 – Runs 1 and 2

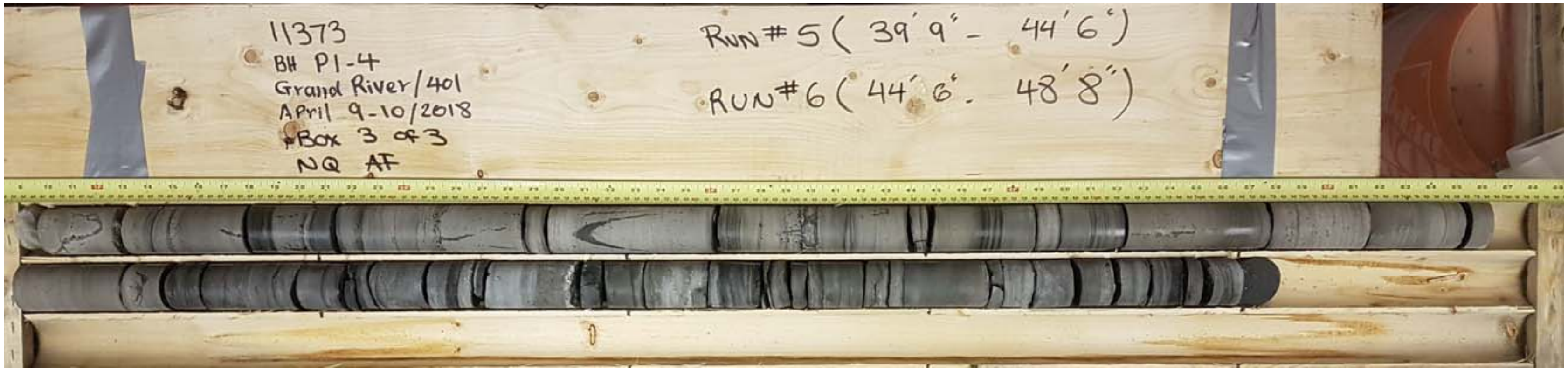
Grand River Bridge Replacement
Photographs of Bedrock Core

Borehole P1-04 – Runs 1 and 2

Borehole P1-04 – Runs 3 and 4

Grand River Bridge Replacement
Photographs of Bedrock Core

Borehole P1-04 – Runs 5 and 6



Grand River Bridge Replacement
Photographs of Bedrock Core

Borehole P2-01 – Runs 1, 2 and 3

Borehole P2-01 – Runs 4, 5 and 6

Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole P2-02 – Runs 1, 2 and 3

Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole P2-03 – Runs 1, 2, 3 and 4

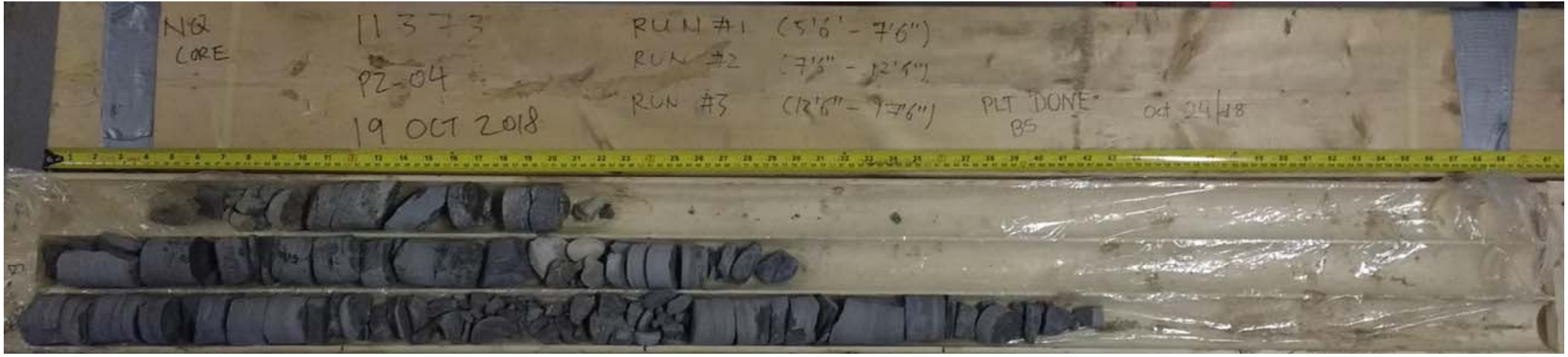
Borehole P2-03 – Runs 5, 6 and 7

THURBER

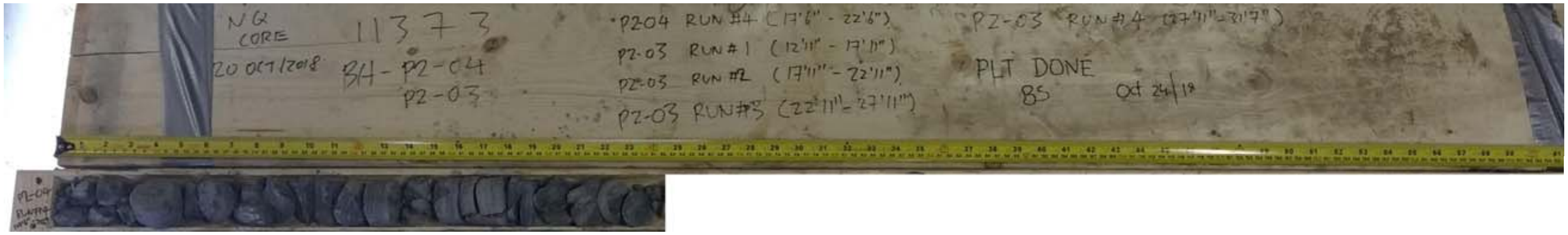
Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole P2-04 – Runs 1, 2 and 3



Borehole P2-04 – Run 4



THURBER

Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole P3-01 – Runs 1 and 2

Grand River Bridge Replacement
Photographs of Bedrock Core

Borehole P3-02 – Runs 1, 2 and 3

Borehole P3-02 – Runs 4, 5 and 6

Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole P3-02 – Run 7

THURBER

Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole P3-03 – Runs 1, 2 and 3

Borehole P3-03 – Run 4

Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole P3-04 – Runs 1, 2 and 3

Borehole P3-04 – Runs 4, 5 and 6

Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole P3-04 – Run 7

Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole EA-01 – Runs 1, 2 and 3

Borehole EA-01 – Runs 4 and 5

Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole EA-02 – Runs 1, 2 and 3

Grand River Bridge Replacement

Photographs of Bedrock Core

Borehole EA-03 – Runs 1, 2, 3, 4 and 5

Borehole EA-03 – Run 6

THURBER

Appendix E

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil	Footings on Bedrock	Driven Piles	Socketed H-Piles
Advantages: i. Relative ease of construction. ii. Shallower excavation depth than footings on bedrock. iii. High resistance values are available at shallow depth at east abutment. iv. Lower cost than deep foundations.	***Advantages:*** i. Relatively high resistance values for footing design. ii. Bedrock surface can be examined to confirm degree of weathering and soundness. iii. Existing bridge is supported on footings on rock, confirming feasibility of system.	***Advantages:*** i. Piles will develop high geotechnical resistance when driven into bedrock or very dense soil. ii. Pile installation may continue in freezing weather. iii. May require less excavation than footing construction.	***Advantages:*** i. High resistance is available for piles socketed into bedrock. ii. Length of pile and socket can be controlled. iii. Avoids pile damage during installation. iv. Construction could continue in freezing weather. v. May require less excavation than footing construction.
Disadvantages: i. Potential large variation in support capability in the valley base deposits. ii. Excavation and cofferdam installation required for footing construction in river valley. iii. Organic and soft soils may extend deeper than anticipated. iv. Potential for undermining by river scour v. Unexpected high river levels may delay construction.	***Disadvantages:*** i. Relatively deep excavation below river water levels, requiring cofferdam installation and significant dewatering. ii. Poor bedrock quality limits design resistance available. iii. Potential for disturbing existing footings. iv. Potential for river scour.	***Disadvantages:*** i. Higher unit cost than footings. ii. Shallow depth to bedrock in the valley base. iii. Difficulty penetrating very dense soil, boulders and rock fragments. iv. Pre-augering may be required. v. Piles may encounter refusal on strong rock layer over weaker weathered layer. vi. Potential for pile damage while driving. vii. Excavation required for pile cap construction.	***Disadvantages:*** i. Higher cost than footings. ii. Difficulty penetrating very dense soil, boulders, rock fragments, and highly weathered bedrock. iii. Temporary steel liners may be required to support side walls above bedrock surface. iv. Grouting of socket around pile is required. v. Difficulty in cleaning and inspecting bases.
FEASIBLE AT EAST ABUTMENT	**FEASIBLE AT PIERS & WEST ABUTMENT**	**NOT RECOMMENDED**	**FEASIBLE**

COMPARISON OF FOUNDATION ALTERNATIVES (cont'd)

Drilled-in Pipe Piles	Caissons	Micropiles
Advantages: i. High resistance is available for piles drilled into bedrock. ii. Avoids potential for meeting refusal on boulders and rock fragments above design tip level. iii. No need for temporary liner. iv. Excavation for pile cap construction below river level could be avoided. v. Construction could continue in freezing weather. ***Disadvantages:*** i. High cost. ii. Relatively deep socket required to contact sound bedrock. iii. Advancing through boulders, rock fragments and highly weathered bedrock may slow production. iv. Limited number of contractors with suitable equipment. **FEASIBLE**	***Advantages:*** i. High resistance is available for caissons socketed into rock. ii. May require less excavation than footing construction. iii. Excavation for pile cap construction below river level could be avoided. iv. Construction could continue in freezing weather. ***Disadvantages:*** v. Higher cost than driven piles. vi. Difficulty penetrating very dense soil, boulders, rock fragments, and highly weathered bedrock. vii. Temporary steel liners may be required to support side walls above bedrock surface. viii. Tremie concrete methods may be required. ix. Difficulty in cleaning and inspecting bases. **RECOMMENDED**	***Advantages:*** i. Lower unit cost than steel piles and caissons. ii. Axial pile resistance may be increased through load tests. iii. May more readily penetrate the boulders and rock fragments than larger diameter foundation types. iv. Pile installation may continue in freezing weather. v. May require less excavation than footing construction. ***Disadvantages:*** i. Very low lateral resistance available unless battered. ii. Prohibitively large number of piles may be required to resist foundation loads. iii. Limited local experience supporting large highway bridges on micropiles. iv. Potential for losing grout into the river through bedrock voids. **FEASIBLE**

THURBER

Appendix F

OPSS/OPSD References and Suggested Wording for NSSPs

1. List of OPSS and OPSD Referenced in this Report

- OPSS.PROV 206 (Construction Specification for Grading)
- OPSS.PROV 212 (Construction Specification for Earth Borrow)
- OPSS.PROV 501 (Construction Specification for Compacting)
- OPSS.PROV 517 (Construction Specification for Dewatering)
- OPSS.PROV 539 (Construction Specification for Temporary Protection Systems)
- OPSS.PROV 804 (Construction Specification for Seed and Cover)
- OPSS.PROV 902 (Construction Specification for Excavating and Backfilling - Structures)
- OPSS.PROV 903 (Construction Specification for Deep Foundations)
- OPSS.PROV 1010 (Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill material)
- OPSD 208.010 (Benching of Earth Slopes)
- OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement)
- SP 105S09
- SP 517F01
- NSSP FOUN0003

2. Suggested Text for NSSP on “Construction of Caissons and Socketed H-Piles”

Caisson and H-pile socket installation shall be in accordance with OPSS.PROV 903. The Contractor is further advised of the following:

- The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating cobbles or boulders in the native soils.

- Caissons and piles will extend through cohesionless soils below the groundwater level. Measures must be employed to maintain sidewall stability in the caisson excavation and prevent collapse/washing of cohesionless soils into the rock socket. Selection of the methods and equipment employed to achieve this is the responsibility of the Contractor.

- The bedrock consists of shale with dolostone and limestone layers. The strength of the bedrock (unconfined compressive strengths of 5 to 250 MPa), and the degree of weathering vary significantly. The strength, hardness and degree of weathering of the bedrock must be taken into account when selecting equipment to advance the socket into rock. Equipment supplied to advance the pile into rock must be capable of penetrating the bedrock without disturbing or fracturing the bedrock adjacent to the caisson. Blasting to facilitate the removal of bedrock is not permitted.

- High volumes of seepage should be anticipated into caisson excavations socketed into bedrock, and measures such as heavy duty pumping to maintain a dry excavation and enable concrete placement in a dewatered condition may not be practical. It is anticipated that placement of concrete using tremie methods will be required.

- After each rock socket is drilled, cleaned and approved, structural concrete must be placed within 24 hours to prevent softening of the shale exposed on the base and sidewalls of the excavation.

3. Suggested Text for NSSP on “Construction of Drilled-in Pipe Piles”

Installation of drilled-in pipe piles shall be in accordance with OPSS 903 and the following.

Drilled-in pipe pile installation at this site will require excavation through very dense sand and gravel, gravelly sand, silty sand, and silty sand till with cobbles and boulders, extending below the groundwater table. The piles must also be advanced into the underlying bedrock containing voids, clay seams, and medium to very strong rock. The Contractor is advised of the following:

- The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating cobbles or boulders in the overburden.

- The strength, hardness and degree of weathering of the bedrock must be taken into account when selecting equipment to advance the pile into rock. Equipment supplied to advance the pile into rock must be capable of penetrating the bedrock to create a clean socket without disturbing or fracturing the bedrock adjacent to the pile. Blasting to facilitate the removal of bedrock is not permitted.

- The annular space between the rock socket wall and pile shall be filled with 30MPa concrete or grout to the top of the bedrock surface. The plumbness and alignment of the pile shall be maintained during concreting.

- During and subsequent to installation, the pipe pile may be partially filled with water and it may not be practical to dewater the pipe prior to concreting. Tremie concreting will be required for concreting these pipe piles.

4. Suggested Text for NSSP on "Installation of Steel Sheet Piles"

Very dense sand and gravel, gravelly sand, silty sand, and silty sand till with cobbles and boulders are present on site. These conditions may impede the driving of sheet piles and at some locations the sheet piles may not be able to penetrate the cobbles and boulders and reach the design depth of installation.

The Contractor shall use appropriate equipment to remove, drill through and/or penetrate these obstructions and extend the piles to the design depth.

Appendix G

Corrosivity Testing – Certificate of Analysis

Certificate of Analysis

AGAT WORK ORDER: 18T337767
PROJECT: Hwy 401 Grand River Bridge 11373

CLIENT NAME: THURBER ENGINEERING LTD
ATTENTION TO: Murray Anderson
SAMPLING SITE:
SAMPLED BY:

Corrosivity Package

DATE RECEIVED: 2018-05-10
DATE REPORTED: 2018-05-17

			SAMPLE DESCRIPTION:	WA-02, SS1, 5-7'	P1-02, SS1, 2.5-4.5'
			SAMPLE TYPE:	Soil	Soil
			DATE SAMPLED:	2018-04-03	2018-04-04
Parameter	Unit	G / S	RDL	9233341	9233344
Sulfide (S2-)	%		0.05	0.07	<0.05
Chloride (2:1)	µg/g		2	42	188
Sulphate (2:1)	µg/g		2	256	35
pH (2:1)	pH Units		NA	7.79	8.86
Electrical Conductivity (2:1)	mS/cm		0.005	0.464	0.377
Resistivity (2:1)	ohm.cm		1	2160	2650
Redox Potential (2:1)	mV		5	178	194

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

9233341-9233344 EC/Resistivity, pH, Chloride, Sulphate and Redox Potential were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).
*Sulphide analyzed at AGAT 5623 McAdam
Samples were received and analyzed beyond recommended hold times.

Certified By: Amanjot Bhela

ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO
CHARTERED
AMANJOT BHELA
CHEMIST

Results relate only to the items tested and to all the items tested

Certificate of Analysis

AGAT WORK ORDER: 18T349377
PROJECT: Hwy 401 - Grand River Bridge 11373

CLIENT NAME: THURBER ENGINEERING LTD
ATTENTION TO: Murray Anderson
SAMPLING SITE:
SAMPLED BY:

Corrosivity Package

DATE RECEIVED: 2018-06-12
DATE REPORTED: 2018-06-19

				SAMPLE DESCRIPTION:		11A - 13+445 2. 5-4.5'		13A - 13+640 5-7'	18A - 14+455 2. 5-4.5'		20A - 14+560 5-7'
				9A - 13+250 5-7'							
			SAMPLE TYPE:	Soil		Soil		Soil	Soil		Soil
			DATE SAMPLED:	2018-05-07		2018-05-07		2018-05-07	2018-05-01		2018-05-02
Parameter	Unit	G / S	RDL	9320989	RDL	9320995	RDL	9320996	9320997	RDL	9320998
Sulfide (S2-)	%		0.05	0.06	0.05	0.06	0.05	0.06	0.06	0.05	0.07
Chloride (2:1)	µg/g		4	489	2	97	4	857	450	2	184
Sulphate (2:1)	µg/g		4	21	2	16	4	19	23	2	15
pH (2:1)	pH Units		NA	10.1	NA	9.67	NA	9.52	10.3	NA	9.51
Electrical Conductivity (2:1)	mS/cm		0.005	1.12	0.005	0.368	0.005	1.73	1.23	0.005	0.572
Resistivity (2:1)	ohm.cm		1	893	1	2720	1	578	813	1	1750
Redox Potential (2:1)	mV		5	230	5	224	5	222	208	5	207

			SAMPLE DESCRIPTION:	30A - 14+805 5-7'		35A - 14+300 5-7'		37A - 14+160 5-7'		B2 - 5-7'
			SAMPLE TYPE:	Soil		Soil		Soil		Soil
			DATE SAMPLED:	2018-04-29		2018-04-29		2018-04-30		2018-04-30
Parameter	Unit	G / S	RDL	9320999	RDL	9321000	RDL	9321001	RDL	9321002
Sulfide (S2-)	%		0.05	0.13	0.05	0.18	0.05	0.05	0.05	<0.05
Chloride (2:1)	µg/g		2	139	4	270	2	122	4	481
Sulphate (2:1)	µg/g		2	128	4	347	2	14	4	26
pH (2:1)	pH Units		NA	9.36	NA	9.07	NA	9.77	NA	9.93
Electrical Conductivity (2:1)	mS/cm		0.005	0.643	0.005	1.28	0.005	0.425	0.005	1.25
Resistivity (2:1)	ohm.cm		1	1560	1	781	1	2350	1	800
Redox Potential (2:1)	mV		5	230	5	227	5	216	5	173

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

9320989-9321002 EC/Resistivity, pH, Chloride, Sulphate and Redox Potential were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).
*Sulphide analyzed at AGAT 5623 McAdam

Samples were received and analyzed beyond recommended hold times.

Elevated RDL indicates the degree of sample dilution prior to the analysis for anions in order to keep analytes within the calibration range of the instrument and to reduce matrix interference.

Certified By: Amanjot Bhela

AMANJOT BHELA

Results relate only to the items tested and to all the items tested

AGAT Laboratories

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

Certificate of Analysis

AGAT WORK ORDER: 18T401987
PROJECT: Hwy 401 - Grand River Bridge

CLIENT NAME: THURBER ENGINEERING LTD
ATTENTION TO: Murray Anderson
SAMPLING SITE:Kitchener
SAMPLED BY:BL

Corrosivity Package

DATE RECEIVED: 2018-10-25
DATE REPORTED: 2018-11-05

			SAMPLE DESCRIPTION:	P2-02, SS3, 7.5-9.5'
			SAMPLE TYPE:	Soil
			DATE SAMPLED:	2018-10-21
Parameter	Unit	G / S	RDL	9654311
Sulfide (S2-)	%		0.05	<0.05
Chloride (2:1)	µg/g		2	26
Sulphate (2:1)	µg/g		2	23
pH (2:1)	pH Units		NA	9.13
Electrical Conductivity (2:1)	mS/cm		0.005	0.136
Resistivity (2:1)	ohm.cm		1	7350
Redox Potential (2:1)	mV		5	196

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

9654311 EC/Resistivity, pH, Chloride, Sulphate and Redox Potential were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).
*Sulphide analyzed at AGAT 5623 McAdam
Pl note: Redox Potential is not an accredited parameter.

Analysis performed at AGAT Toronto (unless marked by *)

Certified By: Nivine Basily

Results relate only to the items tested and to all the items tested

THURBER

Appendix H

Borehole Locations and Soil Strata Drawing

MINISTRY OF TRANSPORTATION, ONTARIO

CONT No
WP No

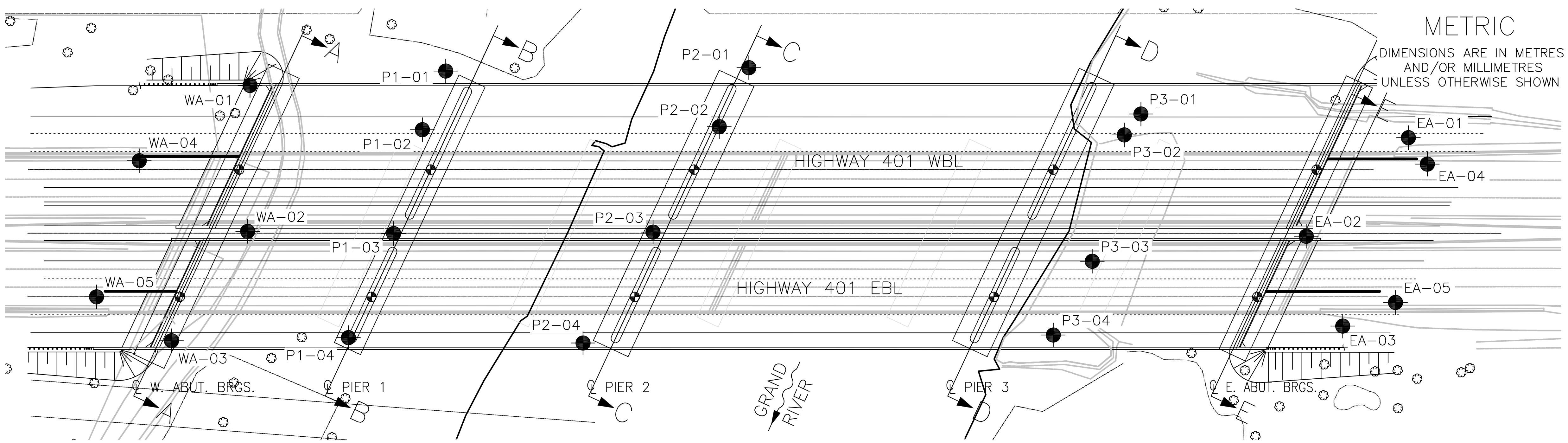
HIGHWAY 401
WESTBOUND LANES
OVER GRAND RIVER
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

wsp

THURBER ENGINEERING LTD.

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



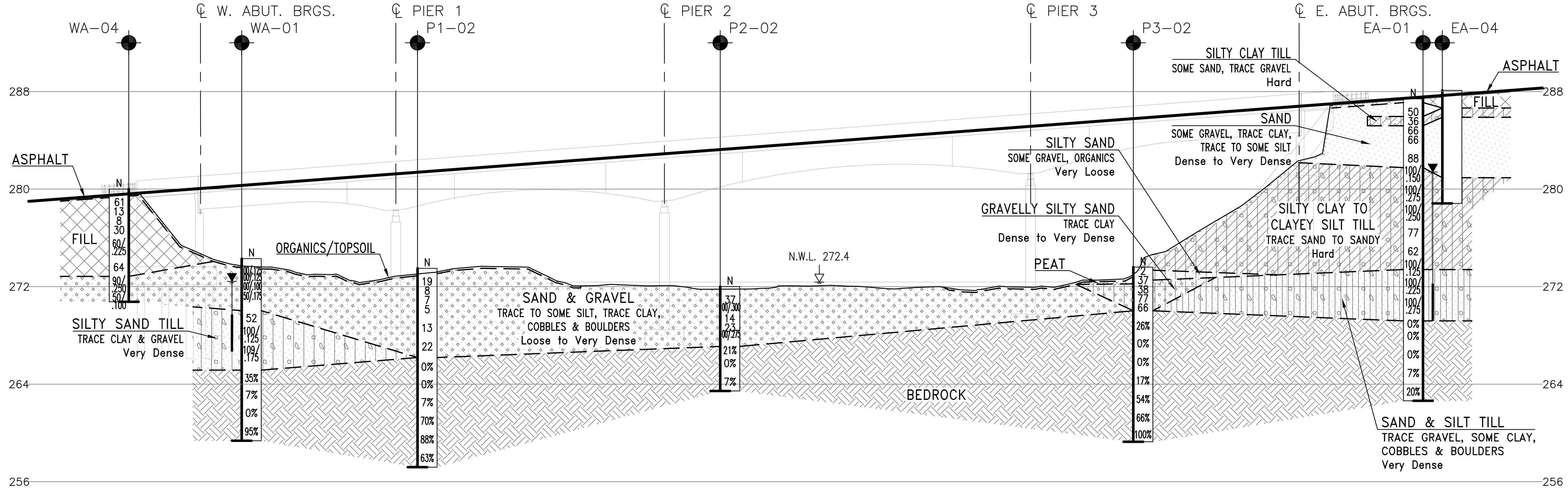
PLAN

20 0 20 40m
SCALE 1:1000

M.R. ANDERSON
90267360
May 12/2020

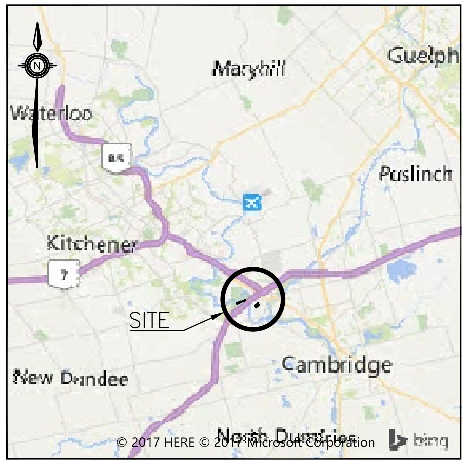
P. K. CHATTERJI
May 12/2020

NO	ELEVATION	NORTHING	EASTING
P2–04	271.9	4 806 790.3	233 000.9
P3–01	274.1	4 806 898.1	233 066.0
P3–02	273.6	4 806 892.6	233 065.8
P3–03	272.5	4 806 867.4	233 076.1
P3–04	273.6	4 806 850.2	233 078.8
WA–01	274.3	4 806 792.0	232 913.1
WA–02	273.8	4 806 767.2	232 930.8
WA–03	274.4	4 806 739.4	232 931.7
WA–04	280.0	4 806 765.6	232 903.9
WA–05	280.0	4 806 737.4	232 913.6



PROFILE ALONG ℄ HIGHWAY 401 WBL

20 0 20 40m H 1:1000
8 0 8 16m V 1:400



KEYPLAN

LEGEND

●	Borehole
⌖	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level During Drilling
▼	Water Level in Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
EA–01	287.4	4 806 927.5	233 113.8
EA–02	282.8	4 806 898.2	233 108.9
EA–03	287.6	4 806 887.7	233 126.2
EA–04	288.1	4 806 925.4	233 120.2
EA–05	287.9	4 806 898.3	233 132.1
P1–01	273.7	4 806 818.7	232 944.1
P1–02	273.5	4 806 806.0	232 947.5
P1–03	272.9	4 806 785.0	232 955.6
P1–04	273.2	4 806 762.0	232 960.9
P2–01	272.1	4 806 857.1	232 994.5
P2–02	272.0	4 806 843.5	232 996.9
P2–03	272.0	4 806 817.6	232 998.9

-NOTES-

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

GEOCRES No. 40P8-246

REVISIONS	DATE	BY	DESCRIPTION

DESIGN MRA	CHK PKC	CODE	LOAD	DATE NOV 2019
DRAWN MFA	CHK MRA	SITE	STRUCT	DWG 1

FILENAME: H:\Drafting\11000\11373\TED–11373–BHPP–GRB.dwg
PLOTDATE: 5/13/2020 3:59 PM

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

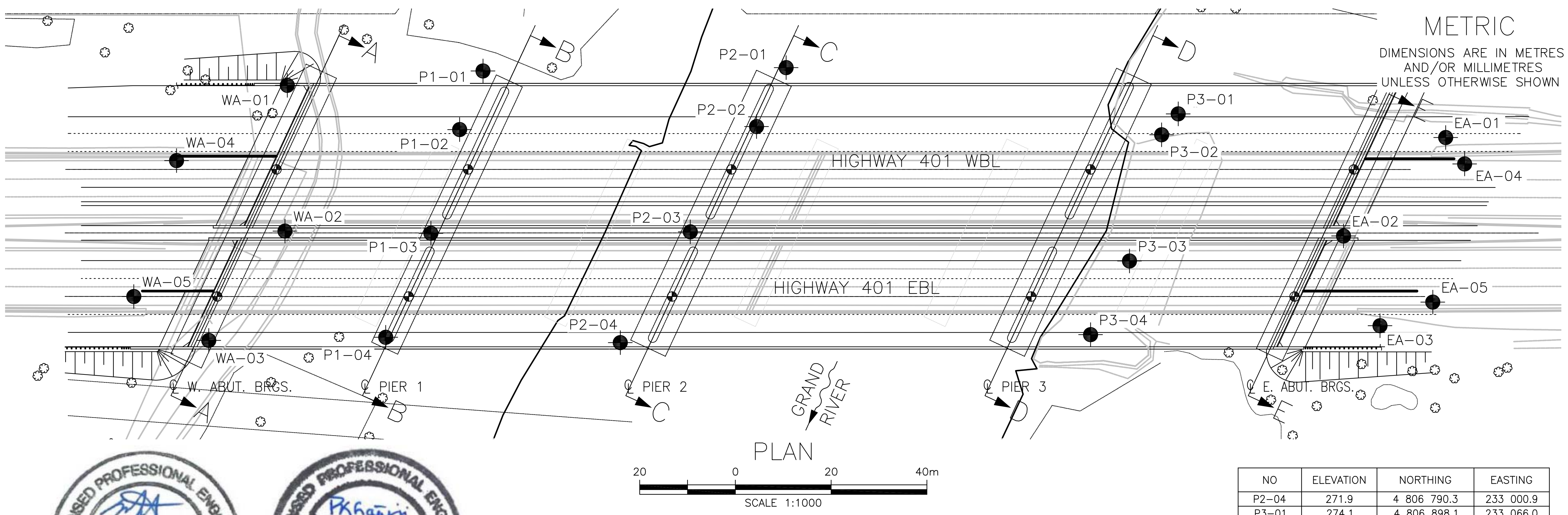
CONT No
WP No

SHEET

HIGHWAY 401
EASTBOUND LANES
OVER GRAND RIVER
BOREHOLE LOCATIONS AND SOIL STRATA

wsp

THURBER ENGINEERING LTD.



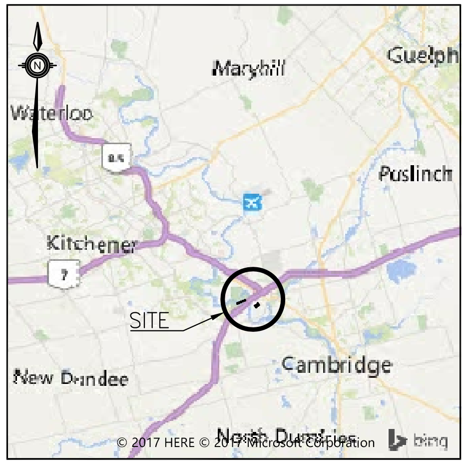
PLAN

SCALE 1:1000

M.R. ANDERSON
90267360
May 12/2020

P. K. CHATTERJI
May 12/2020

NO	ELEVATION	NORTHING	EASTING
P2−04	271.9	4 806 790.3	233 000.9
P3−01	274.1	4 806 898.1	233 066.0
P3−02	273.6	4 806 892.6	233 065.8
P3−03	272.5	4 806 867.4	233 076.1
P3−04	273.6	4 806 850.2	233 078.8
WA−01	274.3	4 806 792.0	232 913.1
WA−02	273.8	4 806 767.2	232 930.8
WA−03	274.4	4 806 739.4	232 931.7
WA−04	280.0	4 806 765.6	232 903.9
WA−05	280.0	4 806 737.4	232 913.6



KEYPLAN

LEGEND

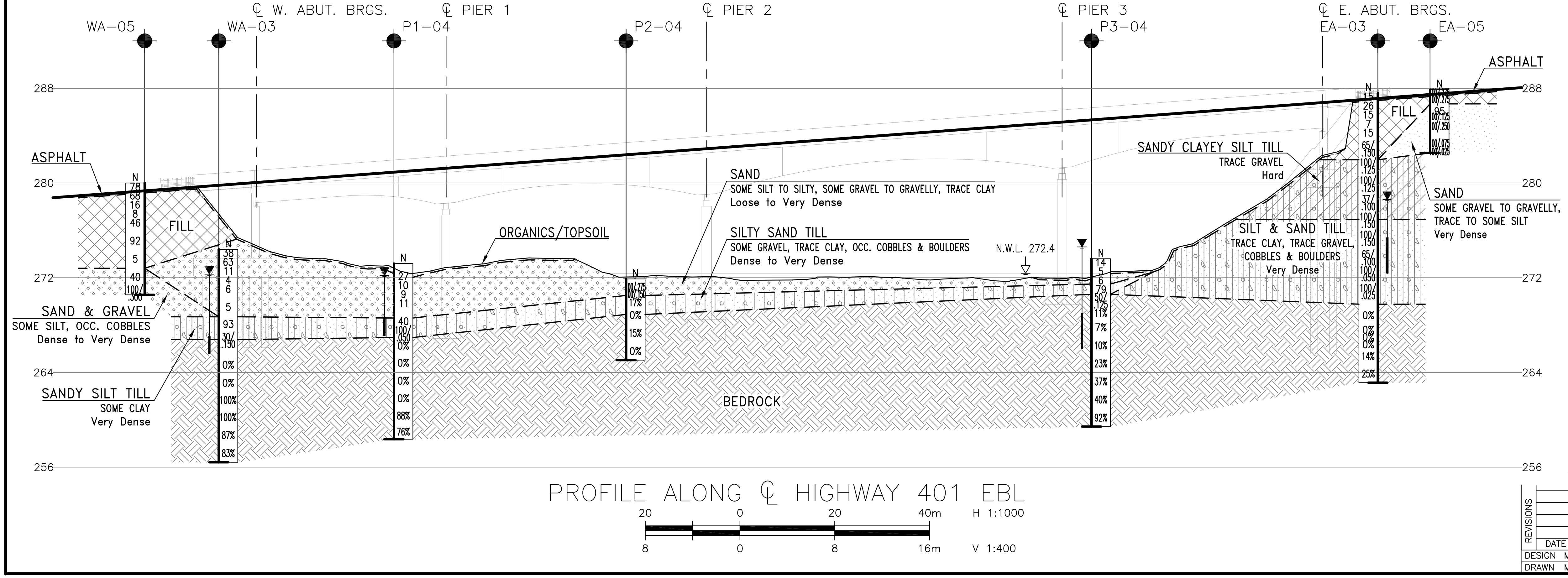
●	Borehole
⊕	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level During Drilling
▼	Water Level in Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
EA−01	287.4	4 806 927.5	233 113.8
EA−02	282.8	4 806 898.2	233 108.9
EA−03	287.6	4 806 887.7	233 126.2
EA−04	288.1	4 806 925.4	233 120.2
EA−05	287.9	4 806 898.3	233 132.1
P1−01	273.7	4 806 818.7	232 944.1
P1−02	273.5	4 806 806.0	232 947.5
P1−03	272.9	4 806 785.0	232 955.6
P1−04	273.2	4 806 762.0	232 960.9
P2−01	272.1	4 806 857.1	232 994.5
P2−02	272.0	4 806 843.5	232 996.9
P2−03	272.0	4 806 817.6	232 998.9

-NOTES-

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

GEOCRES No. 40P8-246



PROFILE ALONG ℄ HIGHWAY 401 EBL

20 0 20 40m H 1:1000

8 0 8 16m V 1:400

REVISIONS	DATE	BY	DESCRIPTION

DESIGN MRA	CHK PKC	CODE	LOAD	DATE NOV 2019
DRAWN MFA	CHK MRA	SITE	STRUCT	DWG 2

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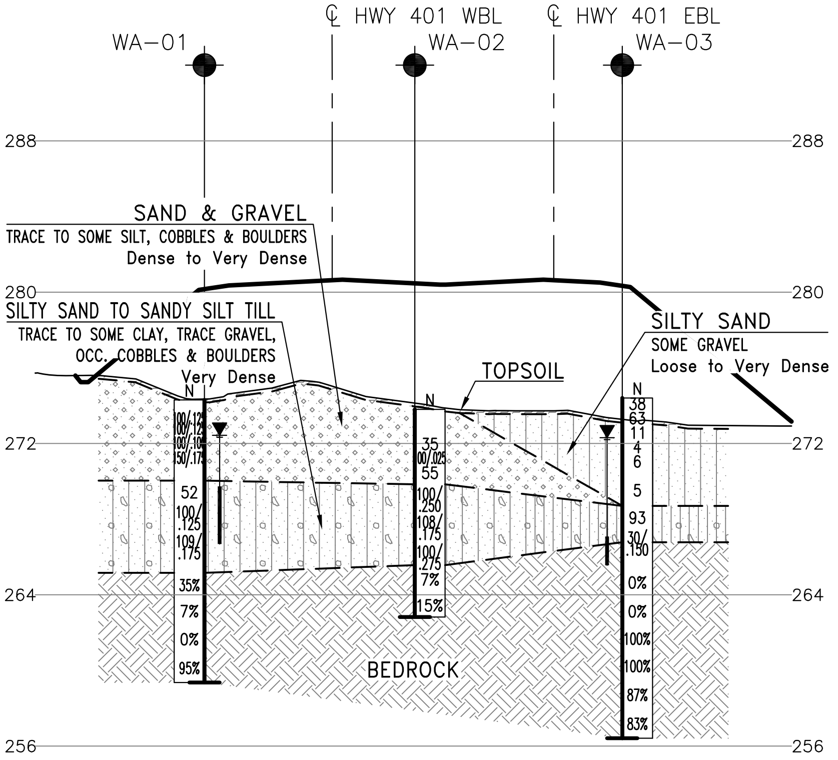
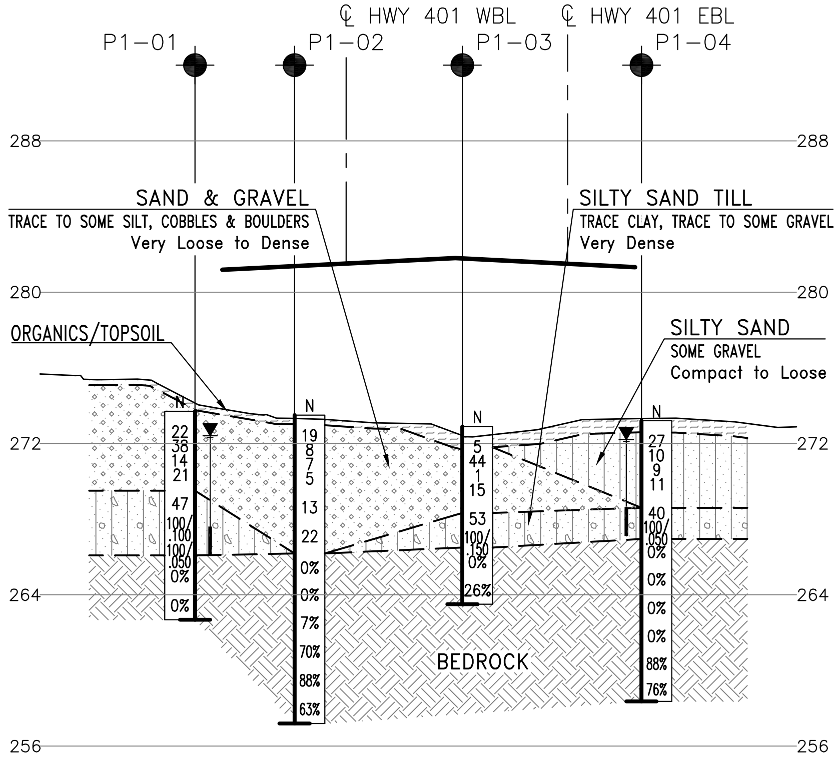
MINISTRY OF TRANSPORTATION, ONTARIO



SECTION A–A (WEST ABUTMENT)



SECTION B–B (PIER 1)

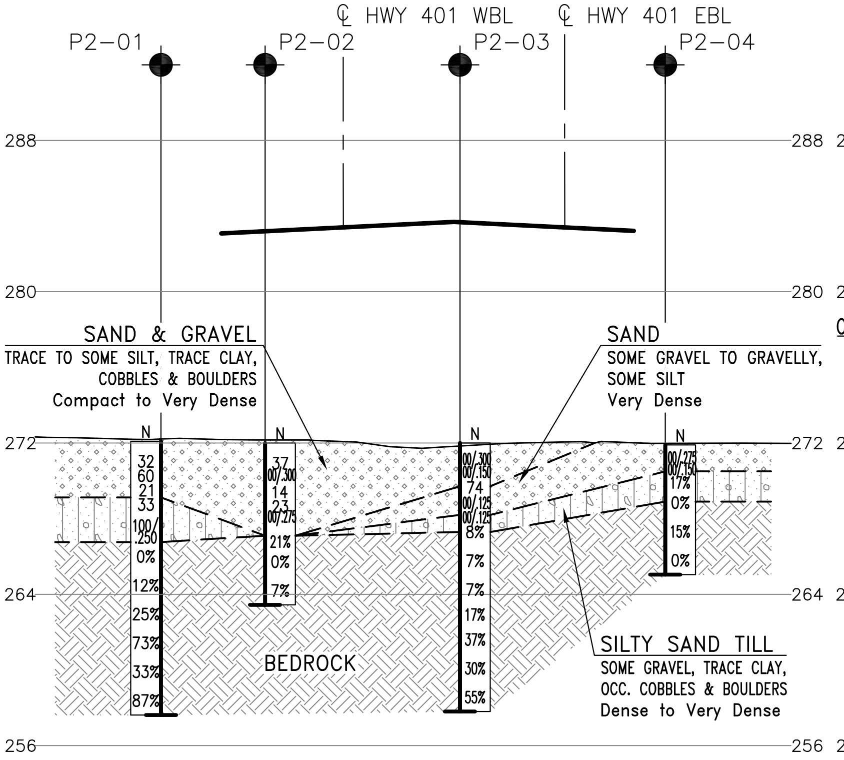
METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

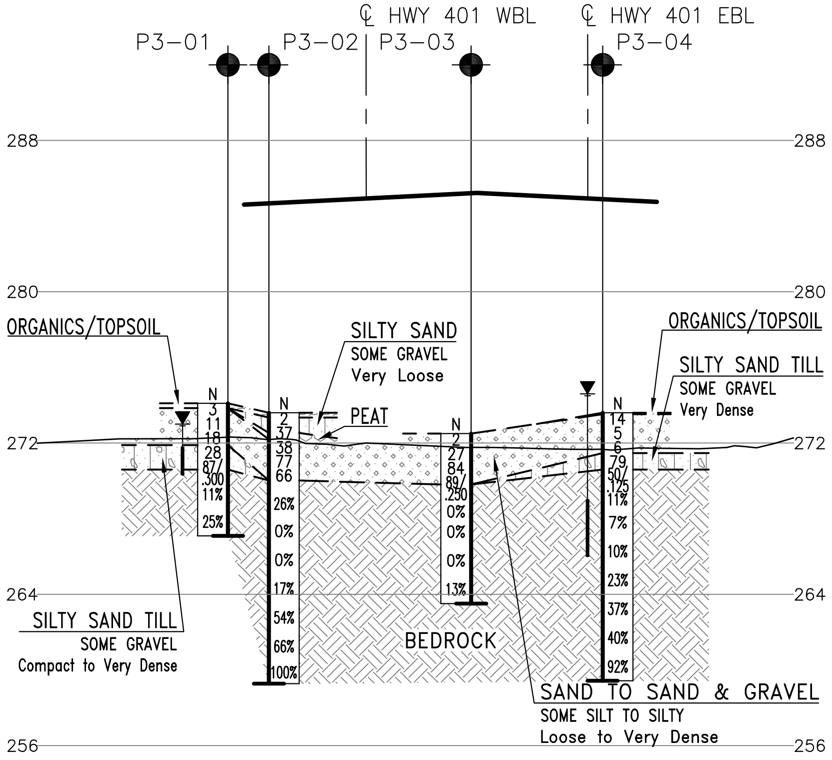
M.R. ANDERSON
90267360
May 12/2020

P. K. CHATTERJI
May 12/2020

NO	ELEVATION	NORTHING	EASTING
P2–04	271.9	4 806 790.3	233 000.9
P3–01	274.1	4 806 898.1	233 066.0
P3–02	273.6	4 806 892.6	233 065.8
P3–03	272.5	4 806 867.4	233 076.1
P3–04	273.6	4 806 850.2	233 078.8
WA–01	274.3	4 806 792.0	232 913.1
WA–02	273.8	4 806 767.2	232 930.8
WA–03	274.4	4 806 739.4	232 931.7
WA–04	280.0	4 806 765.6	232 903.9
WA–05	280.0	4 806 737.4	232 913.6



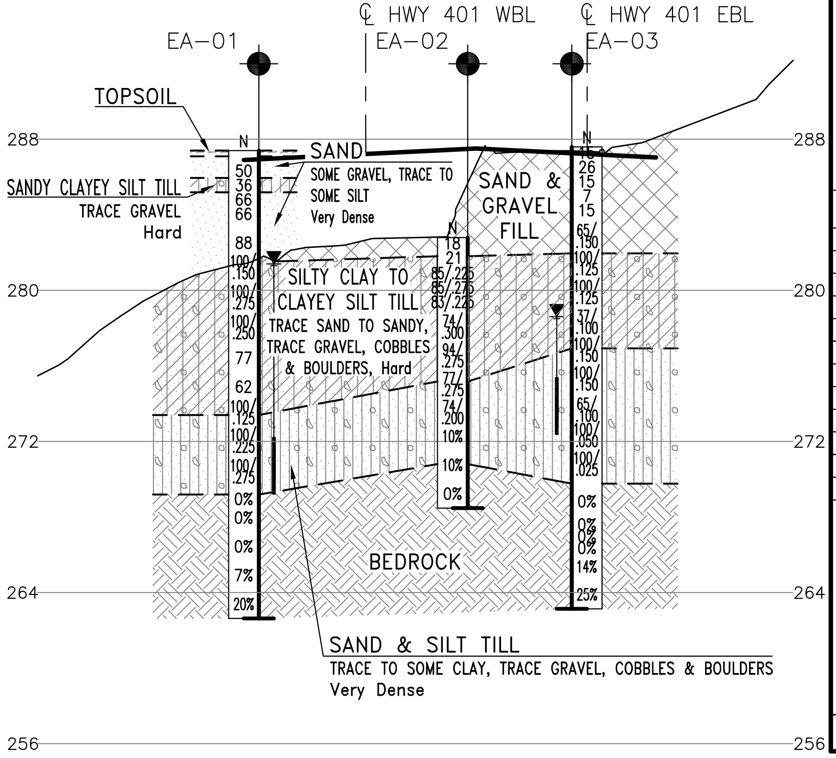
SECTION C–C (PIER 2)



SECTION D–D (PIER 3)

20 0 20 40m H 1:1000

8 0 8 16m V 1:400



SECTION E–E (EAST ABUTMENT)

CONT No

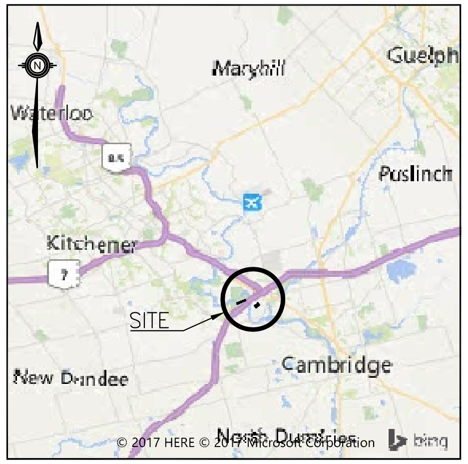
WP No

HIGHWAY 401
EASTBOUND & WESTBOUND LANES
OVER GRAND RIVER
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

wsp

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

●	Borehole
⊕	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level During Drilling
▼	Water Level in Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
EA–01	287.4	4 806 927.5	233 113.8
EA–02	282.8	4 806 898.2	233 108.9
EA–03	287.6	4 806 887.7	233 126.2
EA–04	288.1	4 806 925.4	233 120.2
EA–05	287.9	4 806 898.3	233 132.1
P1–01	273.7	4 806 818.7	232 944.1
P1–02	273.5	4 806 806.0	232 947.5
P1–03	272.9	4 806 785.0	232 955.6
P1–04	273.2	4 806 762.0	232 960.9
P2–01	272.1	4 806 857.1	232 994.5
P2–02	272.0	4 806 843.5	232 996.9
P2–03	272.0	4 806 817.6	232 998.9

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GEOCRES No. 40P8-246

REVISIONS	DATE	BY	DESCRIPTION

DESIGN MRA	CHK PKC	CODE	LOAD	DATE NOV 2019
DRAWN MFA	CHK MRA	SITE	STRUCT	DWG 3

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