APPENDIX

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GEOGTECHNICAL INVESTIGATION REPORT

THURBER ENGINEERING LTD.

PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT
CLASS ENVIRONMENTAL ASSESSMENT STUDY
STEELES AVENUE FROM TREMAINE ROAD TO INDUSTRIAL DRIVE
TOWN OF MILTON, ONTARIO

Report

to

WSP

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APPENDICES

1. INTRODUCTION

This report presents the preliminary geotechnical investigation conducted in support of the Municipal Class Environmental Assessment (EA) Study for improvements to Steeles Avenue between Tremaine Road and Industrial Drive in the Town of Milton, Ontario.

Completion of a Class EA Study is required to assess options for improvements to the Steeles Avenue transportation corridor between Tremaine Road and Industrial Drive, a distance of approximately 1.5 km. Currently the roadway comprises a rural two-lane cross section with a level crossing at the CP railway, a two-lane bridge over Sixteen Mile Creek and a culvert over a tributary of Sixteen Mile Creek. The Region anticipates that required road improvements will include construction of an urban four-lane cross-section realigned south of the existing Steeles Avenue, including a bridge over Sixteen Mile Creek, an underpass at the Canadian Pacific (CP) railway crossing, and a culvert over a tributary of Sixteen Mile Creek approximately 300 m east of Tremaine Road.

The purpose of the preliminary geotechnical investigation was to investigate the subsurface soil and groundwater conditions by means of a limited number of boreholes within the project limits and based on the data obtained, to provide borehole logs, borehole location plans, a written description of the subsurface conditions, and preliminary geotechnical comments and recommendations regarding roadway pavement design, foundations for a railway grade separation, stream bridge and culvert crossings, underground service installations, excavation, and dewatering.

The scope of work for this assignment did not include hydrogeological assessment to evaluate dewatering requirements or environmental assessments, nor a chemical testing program to provide options for reuse or disposal of excavated soil.

Thurber Engineering Ltd. (Thurber) carried out the investigation as a sub-consultant to WSP who are conducting the EA Study for Halton Region.

It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions.

2. BACKGROUND INFORMATION

2.1 Site Description

The proposed Steeles Avenue realignment coincides with the existing alignment for approximately 200 m east of Tremaine Road before extending approximately 300 m south of the existing alignment, crossing through agricultural fields, under the CP railway, through an existing commercial property and over Sixteen Mile Creek before rejoining the existing road alignment approximately 150 m east of Industrial Drive. The total length of the study corridor is approximately 1.5 km.

Steeles Avenue West is an east-west major arterial road with a posted speed limit of 60 km/h. The roadway presently has a two-lane rural cross section with paved shoulders. Realignment and widening of the roadway to a four-lane, major arterial roadway is planned.

Based on a preliminary design drawing provided by WSP titled “Steeles Avenue Option B Underpass” dated June 2020, it is understood the proposed realignment will include the following infrastructure to cross the following features:

- A culvert to cross a seasonal tributary approximately 300 m east of Tremaine Road near station 5+270.
- An underpass with elevated sidewalk at the CP railway between stations 5+855 to 5+885, and;
- A bridge at Sixteen Mile Creek from station approximate 6+115 to 6+145, approximately 270 m west of Industrial Drive.

Selected photographs of the existing conditions observed along the proposed corridor are provided in Appendix A.

2.2 Geology

Based on the information in *The Physiography of Southern Ontario*[1] by Chapman and Putnam (1984), the site is located within the Peel Plain physiographic region. The Peel Plain is characterized by a level to undulating topography gradually sloping towards Lake Ontario with surficial soil comprising a thin lacustrine clay underlain by till.

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[1] Chapman, L.J. and Putnam, D.F. 1984. The Physiography of Southern Ontario, Ontario Geological Survey Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.

Based on *Surficial Geology of Southern Ontario*[2], the surficial material on site is mainly composed of clay and silt till where the materials may have been derived from a glaciolacustrine environment or from the shale bedrock. Older and modern alluvial deposits comprised of clay, silt, sand, and gravel which may contain organic remains are located in the vicinity of Sixteen Mile Creek and the tributary east of Tremaine Road. South of the proposed road alignment are fine textured clay and silt with minor sand and gravel glaciolacustrine deposits with interbedded silt and clay and gritty, pebbly flow till and rainout deposits.

According to *Paleozoic Geology of Southern Ontario*[3], the underlying bedrock geology consists of red shale of the Queenston Formation. The unit is composed of shale and siltstone with minor limestone and sandstone. The bedrock depth is variable due to the undulating topography, however, it is expected to be approximately 2 to 12 m below grade based on well records and drift thickness mapping[4].

2.3 Previous Investigation

Thurber previously carried out a geotechnical investigation for the widening of Tremaine Road, which was documented in the following report:

- Geotechnical Investigation Report, Tremaine Road Widening and Realignment from Main Street to Steeles Avenue in Milton, Ontario by Thurber Engineering for McCormick Rankin (A Member of MM Group Ltd.), Thurber Ref.: 19-1351-235, dated April 23, 2013.

The boreholes from the previous investigation most relevant to current investigation are Boreholes 13-28 and 13-29 located approximately 120 m and 210 m east of Tremaine Road, respectively. Reference is made to the above report for details on the procedures for the field investigation and the results of geotechnical and analytical laboratory testing. The existing borehole data was reviewed and found to provide information regarding the general subsurface conditions in the area, however the boreholes are considered to be located too far away from the proposed culvert to be applicable for culvert foundation design. The applicable borehole locations are shown on the Borehole Location Plans in Appendix B for reference.

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[2] Ontario Geological Survey, 2010: Surficial Geology of Southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 128-REV

[3] Armstrong, D.K. and Dodge, J.E.P., 2007: Paleozoic Geology of Southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 219.

[4] Vos, M.A., Hewitt, D.F., 1969. M2179: Brampton area, southern Ontario, Drift Thickness Sheet, Scale: 1:63 360; Ontario Geological Survey.

The subsurface stratigraphy encountered in these boreholes comprised a pavement structure consisting of 100 mm of asphalt overlying 1,120 mm of sand and gravel granular fill. Silty clay till was contacted below the pavement structure with recorded SPT ‘N’ values of 8 to 34 blows per 0.3 m of penetration, indicating a stiff to hard consistency. Borehole 13-29 was terminated in the clay till at a depth of 2.1 m (Elev. 214.7). In Borehole 13-28, the clay till was penetrated at 4.6 m (Elev. 213.1) and was underlain by clayey silt till to the termination depth of 5.2 m (Elev. 212.5). An SPT ‘N’ value of 29 blows per 0.3 m of penetration was recorded in the silt till, indicating a very stiff consistency. Boreholes 13-28 and 13-29 were open and dry upon completion of drilling.

3. INVESTIGATION PROCEDURES

3.1 Field Investigation

The field investigation for this project was carried out between October 14 and 15, 2020 and comprised a total of three boreholes (Boreholes 20-01 to 20-03) advanced to depths ranging from 4.7 to 10.7 m below ground surface. Borehole details are provided in Table 3.1 and in the Record of Borehole sheets included in Appendix B. The approximate locations of the boreholes are shown on the Borehole Location Plans, Drawings 29750-1 to 29750-2, provided in Appendix C.

Table 3.1 – Borehole Details

Facility	Borehole No.	Ground Elevation (m)	Borehole Termination Depth (m)	Borehole Termination Elevation (m)
Pavement Structure, Municipal Services	20-01	215.9	5.4	210.5
CP Railway Underpass	20-02	211.4	4.7	206.7
Sixteen Mile Creek Bridge	20-03	202.4	10.7	191.7

The borehole locations were established in the field by Thurber using a portable GPS receiver and verified relative to existing site features. The ground surface elevations at the borehole locations were determined using a Trimble R10 GNSS receiver.

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 were advanced using hollow stem augers powered by a track mounted Diedrich D50 drill rig supplied and operated by Walker Drilling Limited. 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). The field investigation was carried out under the full-time supervision of Thurber technical staff. All boreholes were logged in the field. Soil samples were identified, placed in labelled containers, and transported back to Thurber's laboratory in Oakville for further examination and testing.

Groundwater conditions were observed in the open boreholes throughout the drilling operations. Monitoring wells were installed in Boreholes 20-02 and 20-03 to permit monitoring of the groundwater levels at the site. The monitoring wells consisted of 50 mm diameter PVC pipe with a slotted screen sealed at a selected depth within the borehole. The installation details are summarized in Table 3.1 below.

Table 3.1 – Monitoring Well Details

Borehole/ Monitoring Well (BH/MW) No.	Ground Elevation (m)	Monitoring Well Tip		Slotted Screen Length (m)	Mid-Screen Depth (m)	Mid-Screen Elev. (m)
		Depth (m)	Elevation (m)			
20-02	211.4	4.6	206.8	1.5	3.9	207.5
20-03	202.4	10.7	191.7	1.5	10.0	192.4

Borehole 20-01, in which no monitoring well was installed, was backfilled in general accordance with Ontario Regulation 903.

3.2 Laboratory Testing

Geotechnical laboratory testing was carried out at Thurber's laboratory. All recovered soil samples were subjected to visual identification and to natural moisture content determination. Selected samples were also subjected to grain size distribution analysis (hydrometer and/or sieve) and Atterberg Limits testing, where appropriate. Laboratory testing results are summarized on the Record of Borehole sheets included in Appendix B and are presented on the figures included in Appendix D.

4. DESCRIPTION OF SUBSURFACE CONDITIONS

A generalized description of the subsurface conditions encountered in the boreholes is given in the following sections. Detailed descriptions of the soil conditions at the specific locations drilled are presented on the Record of Borehole sheets in Appendix B and take precedence over the generalized description. It should be recognized and expected that soil conditions will vary between and beyond borehole locations.

The subsurface stratigraphy encountered in the boreholes generally comprised of surficial deposits of fill and topsoil, over localized clay deposits, underlain by native deposits of silty clay till and clayey silt till, over clay till/shale complex grading to shale bedrock. Further descriptions of the individual strata are presented below.

4.1 Topsoil

Locally, in Borehole 20-03, a 150 mm thick surficial topsoil layer was encountered and comprised of a silty clay matrix with organic materials. The topsoil thickness will vary between and beyond the borehole location, particularly where trees or large shrubs are encountered, and the reported thickness is not meant to be used for estimating quantities.

4.2 Fill

Silty clay fill was contacted at the ground surface and was penetrated at depths of 1.5 and 2.1 m (Elev. 214.4 and 209.3) in Boreholes 20-01 and 20-02, respectively. Occasional to numerous cobbles and boulders, as well as occasional topsoil inclusions and rootlets, were noted in the fill.

SPT ‘N’ values ranging from 19 blows per 0.3 m of penetration to 50 blows for 0.1 m of penetration were recorded in the silty clay fill layer, indicating a very stiff to hard consistency. Measured moisture contents ranged from 12 to 25%.

The results of a grain size distribution analysis carried out on one sample of the silty clay fill are presented on Figure D1 of Appendix D. The results indicated 0% gravel, 3% sand, 76% silt and 21% clay sized particles.

Atterberg limits testing was carried out on one sample of the clay fill. The measured plastic limit, liquid limit and plasticity index were 19, 30 and 11, respectively. These results, which are plotted on Figure D5 in Appendix D, indicate that the sample tested consists of low plasticity silty clay (CL).

4.3 Clay

In Borehole 20-03, a 1.2 m thick layer of silty clay was encountered below the topsoil and was penetrated at 1.4 m (Elev. 201.0). SPT 'N' values of 22 and 25 blows per 0.3 m of penetration were recorded in the clay layer, indicating a very stiff consistency. It is noted that soil samples from this stratum contained occasional cobbles and rootlets. Moisture contents of 8% and 12% were measured in the clay.

The results of a grain size distribution analysis carried out on a sample of the silty clay are presented on Figure D2 of Appendix D. The results indicated 1% gravel, 20% sand, 55% silt and 24% clay sized particles.

Atterberg limits testing was carried out on one sample of the clay. The measured plastic limit, liquid limit and plasticity index were 19, 37 and 18, respectively. These results, which are plotted on Figure D5 in Appendix D, indicate that the sample tested consists of medium plasticity silty clay (CI).

4.4 Silty Clay to Clayey Silt Till

A silty clay to clayey silt till deposit was contacted below the fill and silty clay deposits in Boreholes 20-01 and 20-03 at depths of and 1.5 and 1.4 m (Elev. 214.4 and 201.0). The till deposit was 0.7 and 5.6 m thick and was penetrated at depths of 2.2 and 7.0 m (Elev. 213.7 and 195.5) in Boreholes 20-01 and 20-03, respectively.

SPT 'N' values recorded in the till deposits ranged from 39 blows per 0.3 m of penetration to 50 blows for 25 mm of penetration, indicating a hard consistency. Measured moisture contents ranged from 4 to 15%.

The results of grain size distribution analyses carried out on selected samples of the silty clay to clayey silt till are shown on Figure D3 in Appendix D. The results of the grain size distribution analyses are summarized below:

Soil Particle	Percentage (%)
Gravel	0 to 3
Sand	10 to 15
Silt	63 to 73
Clay	12 to 24

Atterberg limits testing carried out on a sample of the clayey silt till measured a plastic limit, liquid limit and plasticity index of 21, 15 and 6, respectively. These results, which are plotted on Figure D5 in Appendix D, indicate that the sample tested consists of clayey silt of slight plasticity (CL-ML).

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

4.5 Clay Till/Shale Complex

A zone of clay till/shale complex was encountered below the clay till and fill in Boreholes 20-01 and 20-02. This material typically consists of till with variable amounts of sand, gravel, and shale fragments (to cobble and/or boulder size) and represents the transition between the overlying till deposits and the underlying weathered shale bedrock. It is noted that this material is typically highly variable and can range from non-plastic to plastic. The clay till/shale complex layer was contacted at depths of 2.2 and 2.1 m (Elev. 213.7 and 209.3), was 1.7 and 1.9 m thick and graded to shale bedrock at depths of 3.9 and 4.0 m (Elev. 211.9 and 207.4) in Boreholes 20-01 and 20-02, respectively.

SPT ‘N’ values of 59 per 0.3 m of penetration to 50 blows for 0.1 m of penetration were recorded in the till/shale complex, indicating a hard consistency. Moisture contents of 9 to 11% were measured.

The results of grain size distribution analyses carried out on selected samples of the clay till/shale complex are shown on Figure D4 in Appendix D. The results of the grain size distribution analyses are summarized below:

Soil Particle	Percentage (%)
Gravel	0
Sand	2 to 4
Silt	78 to 80
Clay	16 to 20

Atterberg limits testing carried out on a sample of the clay till/shale complex measured a plastic limit, liquid limit and plasticity index of 27, 18 and 9, respectively. These results, which are plotted on Figure D5 in Appendix D, indicate that the sample tested consists of low plasticity silty clay (CL).

4.6 Shale Bedrock

Shale bedrock was contacted below the clay till/shale bedrock and till in the boreholes. The depth to bedrock and the bedrock surface elevation encountered in the boreholes are summarized in Table 4.5. The boreholes were terminated in the shale bedrock upon practical refusal to auger advance at depths of 4.7 to 10.7 m (Elev. 191.7 to 210.5). SPT N-values of 50 blows per 75 to 100 mm of penetration were recorded. Moisture contents ranged from 5 to 17%.

Table 4.5 – Depth/Elevation of Bedrock Surface

Borehole No.	Bedrock Surface	
	Depth (m)	Elevation (m)
20-01	3.9	211.9
20-02	4.0	207.4
20-03	7.0	195.5

4.7 Groundwater Levels

During drilling, wet conditions were noted in the till material in Borehole 20-03 at an approximate depth of 6.1 m (Elev. 196.3).

The groundwater depths and elevations measured in the monitoring wells installed in the boreholes are summarized in Table 4.6.

Table 4.6 – Summary of Groundwater Level Observations

BH/MW No.	Ground Elev. (m)	Mid-Screen Depth (m)	Mid-Screen Elev. (m)	Groundwater Elevation (metres below ground surface)
				Nov. 3, 2020
20-02	211.4	3.9	207.5	209.7 (1.7)
20-03	202.4	10.0	192.4	201.1 (1.3)

In general, the water level in Borehole 20-03, near Sixteen Mile Creek, is expected to be governed by the prevailing water level in the creek. The water level in Sixteen Mile Creek was recorded at approximate Elev. 202.0 on November 3, 2020.

The above groundwater level measurements are short-term observations and seasonal fluctuations of the groundwater level are to be expected. Further, groundwater and creek water levels may be higher after prolonged periods of precipitation.

5. ENGINEERING DISCUSSION AND RECOMMENDATIONS

This section of the report provides preliminary geotechnical recommendations for design and construction of the roadway improvements and structure foundations. The recommendations are based on the subsurface soil and groundwater conditions encountered during the preliminary investigation. The soil conditions may vary between and beyond the borehole locations. Additional investigation will be required during the detailed design stage to supplement the subsurface information and confirm the preliminary recommendations.

5.1 Pavement Design and Construction

5.1.1 Design Analysis

Steeles Avenue West is an east-west major arterial roadway with a posted speed limit of 60 km/h. The roadway presently has a two-lane rural cross section with paved shoulders. Proposed improvements include realignment and widening of the road to four lanes.

The existing and projected traffic volumes along Steeles Avenue West, provided by WSP, are presented in Table 5.1.

Table 5.1 – Steeles Avenue Traffic Information

Section	Existing AADT (2020)	Future AADT (2031 construction)	Truck Volume	Growth Rate (Linear)
Tremaine Road to Peru Road	10,487	14,534	11.5%	3.51%
Peru Road to Industrial Drive	10,680	16,760	11.5%	5.18%

The traffic data was used to determine the pavement requirements for the anticipated traffic volumes over the design life of the pavement. Using axle load equivalency factors, different axle loads and axle groups are converted to a standard axle load known as an Equivalent Single Axle Loads (ESALs). The Design ESALs calculation was completed in accordance with the MTO *Procedures for Estimating Traffic Loads for Pavement Designs.* Assuming an average truck factor of 1.7, the number of ESALs during a 20-year design period was computed to be 14.6 million.

The pavement design analysis was carried out using the methodology outlined in the 1993 AASHTO "*Guide for the Design of Pavement Structures*", as modified by the Ministry's "*Adaptation and Verification of AASHTO Pavement Design Guide for Ontario Conditions*", and the MTO "*Pavement Design and Rehabilitation Manual*". The AASHTO procedure determines a required Structural Number that characterizes the structural capacity of the pavement layers for a given set of inputs. The following design inputs were used in the AASHTO design analysis.

- Design Period = 20 years
- Initial serviceability, (Pi) = 4.5
- Terminal serviceability (Pt) = 2.5
- Reliability level (R) = 90 percent
- Overall standard of deviation (So) = 0.44
- Mean soil resilient modulus (MR) = 30 MPa

The subgrade for the pavement structure is expected to consist primarily of very stiff to hard silty clay fill or native clay, silty clay till or clayey silt till. This subgrade condition must be confirmed along the entire alignment prior to final design.

Based on the design input parameters and calculated ESALs, a design structural number (SN_{Des}) of 150 mm is required. The recommended pavement design thickness, based on the structural requirements, traffic projections, and subgrade conditions, is presented below.

5.1.2 Recommended Preliminary Pavement Design

Based on the borehole data, the anticipated traffic volumes, and assuming adequate subgrade drainage, the following preliminary pavement design is recommended for realignment and widening of Steeles Avenue West:

Component	**Thickness**
HL1	50 mm
HDBC (2 lifts)	130 mm
OPSS Granular A Base	150 mm
OPSS Granular B Type II Subbase	500 mm

The pavement design thicknesses should be reviewed during detailed design.

The minimum PGAC grade of virgin asphalt cement in the surface and top binder course should be PG 64-28, and minimum PG 58-28 for the lower binder course. Consideration should be given to further upgrading of the PGAC grade to PG 70-28 if rutting has been experienced in other sections of this roadway due to truck traffic. Aggregates for the asphalt mixes should be in accordance with OPSS.MUNI 1003.

Should the Region consider using Superpave asphalt mixes for this project, the recommended HL1 material should be substituted with a Superpave 12.5 FC1 asphalt mix, and the HDBC asphalt material should be replaced with Superpave SP 19. As the 20-year design ESALs were estimated to be 14.6 million, a Traffic Category D designation should be used in preparing all Superpave asphalt mix designs.

All new granular subbase material should consist of OPSS Granular B Type II, while the granular base material should consist of OPSS Granular A. All new granular material should meet the requirements of OPSS 1010, and be compacted to 100 percent of the Standard Proctor Maximum Dry Density (SPMDD) within 2 percent of Optimum Moisture Content (OMC). All granular material should be compacted in accordance with the requirements of OPSS.MUNI 501, and should be carried the entire width of the roadway platform to maintain appropriate drainage.

5.1.3 Pavement Subgrade Preparation

Pavement subgrade preparation should include removal of any existing pavement structure and all surficial vegetation, topsoil, organic or compressible material. The exposed subgrade should be compacted and proof-rolled with a heavy roller and examined by an experienced Geotechnical Engineer to identify areas of unstable subgrade. Any soft/wet areas identified shall be subexcavated and replaced with approved material within 2% of Optimum Moisture Content (OMC), and compacted to at least 98% of Standard Proctor Maximum Dry Density (SPMDD).

Bulk fill used to raise the road grade should be constructed as engineered fill, consisting of approved inorganic material, placed in maximum 200 mm thick lifts, within 2% of optimum moisture content, and compacted to at least 98% of SPMDD. Standard side slopes of 2H:1V or flatter should be suitable for embankment construction. Exposed embankment surfaces should be provided with a vegetation cover or otherwise protected against erosion in accordance with OPSS 804.

The top of the compacted subgrade should be graded smooth with a minimum crossfall of 3% towards subdrains. Continuity of drainage should be maintained at transitions from existing pavement to new pavement.

5.2 Preliminary Foundation Design

5.2.1 Sixteen Mile Creek Bridge

Based on a drawing by WSP titled “Steeles Avenue Option B Underpass” dated June 2020, the proposed Steeles Avenue West alignment will cross Sixteen Mile Creek from approximate station 6+115 to 6+145, approximately 270 m west of Industrial Drive. It is understood that the bridge span length has not been determined at the time of this report.

The subsurface stratigraphy encountered in Borehole 20-03, drilled to the south of Sixteen Mile Creek, consisted of 0.2 m of topsoil, overlying a 1.2 m thick very stiff clay layer, underlain by a 5.6 m thick deposit of hard silty clay to clayey silt till, mantling shale bedrock at a depth of 7.0 m. A groundwater level was measured in the monitoring well in Borehole 20-03 at a depth of 1.3 m (Elev. 201.1). It is noted that occasional to numerous cobbles and boulders were encountered in the borehole.

Based on the borehole data, the preferred means of supporting the bridge comprises either spread footings on the native hard clay/silt till, or deep foundations consisting of either driven h-piles or augered cast in place caissons extended to bedrock.

For preliminary assessment of the bridge design, factored geotechnical resistances of 500 kPa at ULS and 350 kPa at SLS are recommended for preliminary design of spread footings on the hard native till founded at or below Elev. 201.0 plus any scour depth required. Excavation for footing construction would need to extend through the surficial silty clay and into the till below the creek water level. Cofferdam installation and/or advance dewatering may be necessary to enable construction of footings in the dry near the creek.

A foundation system comprising augered caissons socketed into shale bedrock may be considered for the Sixteen Mile Creek bridge if higher foundation capacities are required. The recommended axial geotechnical resistances at factored ULS for 1.2 and 1.5 m diameter caissons socketed a minimum of 4 m into shale bedrock are 4,500 and 6,000 kN, respectively. The geotechnical resistance at SLS is not expected to govern design. The resistance values are based on the assumption that the walls and base of each caisson are cleaned of loose material prior to placement of concrete.

The axial capacities provided are for preliminary design only and must be reviewed during detail design when rock coring of the bedrock has been completed.

The geotechnical resistance values assume a minimum centre-to-centre caisson spacing of three caisson diameters. The resistance values may need to be reduced for lesser caisson spacing.

The installation of caissons may be impacted by occasional boulders, the possible presence of cohesionless layers within the till deposit, and a high groundwater level. Construction may require use of a steel liner to maintain stability of the caisson sidewalls.

Shale bedrock generally becomes harder/more sound with depth and contains hard siltstone or limestone interbeds. The presence of the hard layers may slow auger advance or require use of coring equipment during socketing of the caissons. The caisson drilling equipment selected by the contractor must be capable of advancing through the hard layers.

Supporting the bridge on steel H-piles driven into the shale bedrock may be feasible, however, due to the occasional to numerous cobbles and boulders encountered in the boreholes, there may be installation difficulties. If piles are used, it is anticipated that pre-drilling will be required.

It is recommended the final decision on foundation type be reviewed during detailed design when more information is available.

5.2.2 CPR Crossing Underpass

It is understood that grade separation at the CP railway is planned. Current plans call for the construction of an underpass structure located between stations 5+855 and 5+885. Based on a drawing provided by WSP, it is understood that Steeles Avenue will pass under the CP railway at elevation 202.8, approximately 9 m below existing grade.

The stratigraphy encountered in Borehole 20-02 drilled near the CP crossing consisted of very stiff to hard silty clay fill to a depth of 2.1 m (Elev. 209.3), underlain by hard clay till/shale complex grading to shale bedrock at 4.0 m (Elev. 207.4). A groundwater level was measured at a depth of 1.7 m (Elev. 209.6) in the monitoring well.

Based on the borehole data, the underpass will be constructed approximately 5.0 m into the shale bedrock.

Considering the presence of shale bedrock at the base of the road cut, the preferred foundation system for the structure consists of spread footings bearing on rock. Caissons socketed into the rock could also be considered if higher foundation capacities are required.

Supporting the rail bridge structure on spread footings founded on shale bedrock is considered feasible. Factored geotechnical resistances of 1,000 kPa at ULS may be employed for preliminary design of spread footings on the sound shale bedrock below Elev. 203.0. The geotechnical resistance at SLS is not expected to govern design of spread footings on shale bedrock. For working stress design (AREMA code), an allowable bearing capacity of 1,000 kPa is recommended. The provided capacities should be confirmed with additional rock coring and analysis during detailed design.

Augered caissons will develop axial resistance through a combination of sidewall shear and end bearing in the rock socket. The recommended axial geotechnical resistances for 1.2 and 1.5 m diameter caissons socketed a minimum of 4 m into shale bedrock are 4,500 and 6,000 kN at factored ULS, respectively. The geotechnical resistance at SLS is not expected to govern design. The resistance values are based on the assumption that the walls and base of each caisson are cleaned of loose material prior to placement of concrete. For working stress design (AREMA code), allowable capacities of 3,000 and 4,000 kN are recommended for similarly socketed caissons with diameters of 1.2 and 1.5 m, respectively.

Reference is made to the previous section for additional comments and recommendations for design and construction of caissons.

5.2.3 Tributary Culvert

Construction of a culvert over a tributary located approximately 300 m east of Tremaine Road is planned near station 5+270.

Boreholes 13-28 and 13-29 from the previous investigation of Tremaine Road are the closest boreholes to the proposed culvert. The boreholes were drilled on Steeles Avenue West and are considered too far away to provide an accurate representation of the subsurface conditions at the culvert location. Therefore, the comments presented below regarding preliminary foundation design, based on the conditions encountered in the two boreholes, do not necessarily reflect the actual conditions at the location of the culvert. A detailed drilling program will be required to confirm conditions at the proposed structure location.

The subsurface stratigraphy encountered in Boreholes 13-28 and 13-29 comprised a pavement structure over stiff to hard silty clay till underlain by hard clayey silt till. The boreholes were open and dry upon completion of drilling.

For preliminary assessment of the culvert design, factored geotechnical resistances of 225 kPa at ULS and 150 kPa at SLS are recommended for preliminary design of spread footings on the stiff to hard, native silty clay/clayey silt till.

Higher resistances may be achieved at greater depths if required for open footing design. Based on the borehole data, consideration may also be given to supporting the proposed structure on driven pile foundations, or augered caissons. The preferred alternative for the foundations may depend upon the subsurface conditions specific to that foundation location and will need to be determined/confirmed during detailed design.

All surface vegetation, topsoil, organic deposits, disturbed material, or otherwise loose/soft soils must be stripped from the culvert area prior to culvert installation. Inspection and approval of the subgrade by geotechnical personnel is recommended prior to placement of bedding material.

Bedding and backfill to the culvert should be in accordance with OPSD 803.010. A minimum 300 mm thickness of Granular A bedding material is recommended below the culvert. The bedding thickness may need to be increased where subexcavation is required to remove deleterious materials below the design excavation level or a less competent subgrade is encountered.

Where headwalls are provided, horizontal resistance against sliding may be developed by frictional resistance between the concrete footing and the underlying clay till. For cast-in-place concrete on stiff to hard clay till, an ultimate friction factor of 0.5 is recommended. A suitable safety factor should be applied to this value.

5.2.4 Seismic Design 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 a dense/hard overburden layer overlying shale bedrock. As per Table 4.1, Clause 4.4.3.2 of the CHBDC, the site may, on a preliminary basis, 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 0.118 g at the site.

In order to confirm the seismic site class for each structure in accordance with the CHBDC, where required for final design, a borehole must be installed to a minimum of 30 m depth below grade to confirm ground conditions, or ground shear wave velocity to 30 m depth confirmed using geophysical testing methods, such as the Multichannel Analysis of Surface Waves (MASW) method.

5.2.5 Frost Cover

The depth of frost penetration at this site is approximately 1.2 m. All spread footings or pile caps should be provided with a minimum of 1.2 m of earth cover as protection against frost action.

5.3 Excavations and Groundwater Control

Excavations for construction of the bridge and culvert foundations are anticipated to depths of 3 to 5 m below existing grade and will extend through the surficial silty clay fill and native silty clay and into the very stiff to hard till below the creek and tributary water levels. Excavations for construction of the CP underpass is proposed to depths of 8 to 9 m below existing grades and is anticipated to extend through the silty clay fill and clay till/shale complex deposits and into the shale bedrock. Excavations to these depths are expected to extend up to 7 to 8 m below the measured groundwater levels.

All excavations should be carried out in accordance with the requirements of the Occupational Health and Safety Act (OHSA) and local regulations. For the purposes of the OHSA, the soils within the likely depth of excavation at these sites may be classed as Type 3 soils for very stiff silty clay fill. The native stiff to hard silty clay and silty clay till and till shale complex may be classed as Type 2 soil. Where space restrictions preclude excavation of inclined slopes, excavation may be carried out using a trench box or temporary shoring.

Use of a hydraulic excavator should be suitable for excavation in the fill and native 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. The fill, clay, and till deposits contain cobbles and boulders and may also contain other obstructions and the contractor must be prepared to handle these obstructions.

The upper 1 to 3 m of the Queenston shale formation is typically weathered and excavation should be possible using heavy excavation equipment and rippers, supplemented by pneumatic rock breakers where thick layers of hard material are encountered. The shale below this depth is harder and less weathered, and intensive use of pneumatic/hydraulic breakers, line drilling or other methods of loosening the bedrock will likely be required.

Near vertical sidewalls may be employed in unweathered shale bedrock, subject to geotechnical inspection at time of construction. Some scaling back and flattening of the bedrock face may be required particularly in zones of concentrated seepage and areas of loose rock fragments and/or slaked material that develops over time.

Seepage into excavations should be anticipated where excavations will extend below the observed water levels and measures such as heavy-duty pumping and/or perimeter wells may be required to maintain a dry excavation. Concentrated seepage may be experienced from seams or fractures in the shale bedrock. Stream flow and surface water runoff must be diverted away from the excavations at all times during construction.

Considering the subsurface conditions encountered in the boreholes (relatively impermeable clay and till deposits grading to shale bedrock) it is possible that the dewatering for shallow foundation excavations and deeper underpass excavations using sumps and pumps may be feasible.

As the underpass will be approximately 6 to 7 m below the groundwater table, temporary groundwater control during construction and possible long term permanent groundwater control following construction will be required. The structure must be designed as either a drained structure, with permanent dewatering to a positive drain, or an undrained structure (bathtub).

Effective dewatering operations rely on the Contractor's experience, construction techniques, sequencing, and work force efficiency.

Groundwater control must be the responsibility of the contractor. The contractor must retain a dewatering specialist to design the dewatering system and identify effective measures for the conditions encountered. The dewatering plan should be submitted for information purposes before the start of excavation. The impact of the dewatering on local water wells or other groundwater resources in the area would need to be assessed prior to adopting this method of construction.

It is recommended that a hydrogeological assessment is completed during detailed design to determine the anticipated dewatering rates, and assessment of impacts resulting from dewatering, including possible mitigations. If the anticipated dewatering rates range between 50,000 and 400,000 L/day, the water taking must be registered on the Ministry of the Environment, Conservations and Parks (MECP) Environmental Activity and Sector Registry (EASR). A Permit to Take Water (PTTW) will be required if pumping rates are expected to exceed 400,000 L/day.

It is noted that ground water sampling and chemical testing was not within the scope of this investigation. Sampling and testing of the ground water during detailed design will be required to provided discharge options.

5.4 Abutment Backfill and Lateral Earth Pressures

Backfill behind the culvert, grade separation structure and bridge abutments should consist of non-frost susceptible, free-draining granular material conforming to OPS Granular A or Granular B Type II specifications.

The lateral earth pressures acting on the walls, assuming full drainage from behind the walls, may be calculated from the following expression:

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

Where: p_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see table 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)

Table 5.4 lists unfactored parameters for design purposes, assuming an essentially level ground surface behind and in front of the walls.

Table 5.4: Unfactored Earth Pressure Parameters

Retained Material	Unit Weight (kN/m^3)	Friction Angle (degrees)	Earth Pressure Coefficient		
			Active (K_a)	At-rest (k_o)	Passive (K_p)
Granular A or B Type II	22.8	35	0.27	0.43	3.7
Granular B Type I	21.2	32	0.31	0.47	3.3

If lateral movement is not permissible and/or the wall is restrained from lateral yielding, the at-rest earth pressure coefficient, K_o, should be used. If the wall design allows lateral yielding (non-rigid structure), the active earth pressure coefficient, K_a, may be used.

The earth pressure coefficients in the table above do not include potential compaction effects that must be included in the design. Compaction effects should be considered as per the CHBDC.

Design of the structures must incorporate measures such as weepholes to permit drainage of the backfill and avoid potential build-up of hydrostatic pressures behind the walls.

5.5 Embankments and Retaining Walls

Based on the preliminary profile drawings of the alignment provided by WSP, fill embankments and retaining walls will be required in association with each of the structures. Preliminary details based on provided drawings are as follows:

- The approach embankments to the proposed tributary creek culvert structure will be up to 3 m and extend approximately 35 and 40 m west and east of the culvert, respectively.
- The proposed roadway will be in a cut from approximately 300 m south to 100 m north of the proposed CP railway underpass. It is anticipated that retaining walls up to 9 m high and running parallel to the roadway will be required.
- The approach embankments for the proposed Sixteen Mile Creek Bridge will be up to 2 m in height and extend approximately 60 m west and east of the abutments.
- Additional embankments and retaining walls may be required along the roadway where grades are proposed to be raised or lowered from existing grades.

Preliminary comments regarding the anticipated foundation conditions, stability and settlement of the fill embankments and retaining walls are presented below.

Embankments

The foundation soils underlying the proposed approach embankments are expected to consist primarily of very stiff to hard clay fill to depths of 1.5 to 2.1 m and very stiff native clay and stiff to hard till deposits. Fill materials are not considered feasible for embankment founding soils and must be excavated to the level of native soils and replaced with approved embankment fill material. In general, the stability of embankment slopes and settlement of the native foundation soils under the embankment loads are not expected to be a concern.

Embankments with standard side slope inclinations of 2H:1V are expected to be stable. Mid-height berms comprising 2 m wide benches must be incorporated along the length of embankments with heights exceeding 6 m. Where new embankment fill is placed on a sloping ground surface, the existing earth slope must be benched in accordance with OPSD 208.010. Earth fill embankment slopes must be provided with erosion protection in accordance with OPSS.PROV 804.

Retaining Walls

The retaining structures at the proposed CP rail underpass may be founded on either hard native clay till or shale bedrock. Spread footings constructed on sound shale at the base of the cut may be designed using a factored geotechnical resistance of 1,000 kPa at ULS, as per the preliminary foundation recommendations provided for the grade separation structure in Section 5.2.2. As roadway grades rise out of the base of the cut, the design founding levels will rise and footings may be founded on till and weathered bedrock. For preliminary design of spread footings founded within the upper 2.0 m of shale or on hard native till at or below Elev. 209.0, factored geotechnical resistances of 500 kPa at ULS and 350 kPa at SLS are recommended.

All loose, disturbed, or wet material should be removed from the base of the wall excavations. The founding surface should be inspected by a geotechnical engineer to confirm that the exposed surface has been adequately prepared and that the conditions are as interpreted in the design.

Backfill behind the retaining wall should consist of free-draining Granular A or Granular B material. The lateral earth pressures acting on the walls, assuming full drainage behind the walls, may be computed as described in Section 5.4. The design should include drains and the base of the walls, leading to a positive outlets.

The design of the walls should take into consideration any additional surcharge loading due to surface grades, temporary stockpiles, or vehicles.

5.6 Municipal Service Installation

In general, excavation for open cut installation of municipal services will extend through the existing fill materials and native clay, into native till and till/shale complex deposits and shale bedrock.

Excavation and groundwater control should be in accordance with the recommendations in Section 5.3.

Prior to placement of pipe bedding, the base of the trench should be maintained in a dry condition, free of loose or disturbed material. The pipe must be placed on a uniformly competent subgrade. Pipe bedding materials, compaction and cover should follow OPSD 802.030 to 803.034, and/or Halton Region specifications.

Trench backfill materials should be placed in loose lift thicknesses not exceeding 200 mm and compacted to at least 98% of its SPMMD. Where utility trenches are located beneath the roadway, OPSS Granular A or B material, or unshrinkable fill should be employed as backfill.

For trenches located outside of the roadway, the portion of the trench above the pipe cover can be backfilled with excavated soil provided it is unfrozen and free of organics, debris and other deleterious materials. The placement moisture content should be within about 2% of the optimum moisture content for efficient compaction, and the till must be adequately broken down and compacted in the trench.

5.7 Detailed Investigations

The information presented in this report is provided for preliminary design and planning purposes only. Detailed geotechnical investigation will be required to confirm the subsurface conditions and recommendations. This work should incorporate:

- A detailed pavement investigation including additional boreholes within the proposed roadway alignment areas to further define the subgrade conditions and confirm the pavement design recommendations;
- Boreholes within the envelope of all foundation units to confirm the subsurface conditions at the structure locations and develop detailed geotechnical recommendations for design and construction of the new grade separation structure, culvert, and bridge foundations;
- Additional investigation along the proposed retaining walls, fill embankments, and temporary roadway protection locations;
- Further assessment of dewatering requirements, impacts and mitigations, and the need for a PTTW or EASR; and
- Environmental site assessments and chemical testing of soils in accordance with O.Reg. 406/19, to determine management options for excess excavated soils.

STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. THURBER IS NOT RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT THURBER'S WRITTEN CONSENT AND SUCH USE SHALL BE ON SUCH TERMS AND CONDITIONS AS THURBER MAY EXPRESSLY APPROVE. Ownership in and copyright for the contents of the Report belong to Thurber. Any use which a third party makes of the Report, is the sole responsibility of such third party. Thurber accepts no responsibility whatsoever for damages suffered by any third party resulting from use of the Report without Thurber's express written permission.

5. INTERPRETATION OF THE REPORT

a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.

d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

7. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.

Appendix A

Site Photographs

Photograph 1 – View of agricultral field at western project limits from Tremaine Road looking east.

Photograph 2 – View of agricultral field and Steeles Avenue West at eastern project limits looking west.

Photograph 3 – View of agricultral field looking north along the proposed alignment between Boreholes 20-01 and 20-02.

Photograph 4 – View of CP railway at location of proposed underpass looking northeast.

Photograph 5 – View of gravel parking lot at proposed alignment north of CP railway looking southwest.

Photograph 6 – View of proposed alignment near Borehole 20-03, south of Sixteen Mile Creek, looking north.

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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.

Previous Investigation

RECORD OF BOREHOLE 13-28

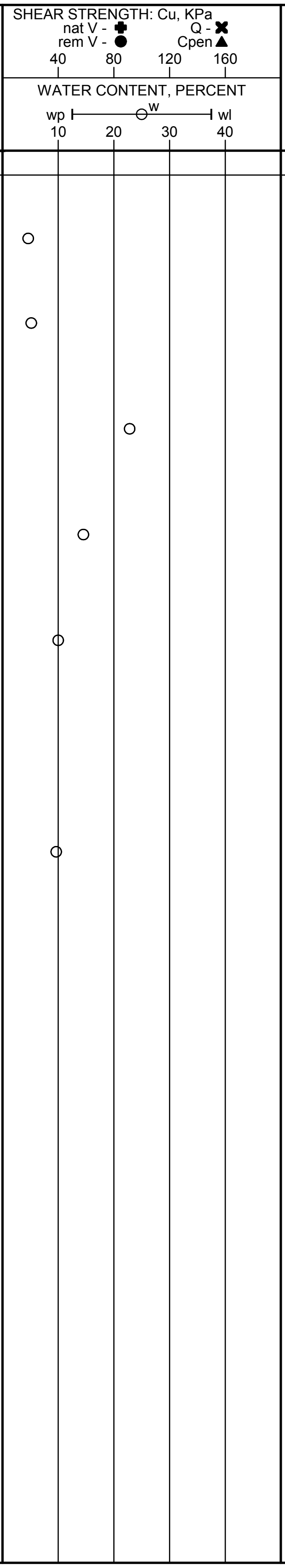
PROJECT : Tremaine Road - Main Street to Steeles Avenue
LOCATION : Milton, ON
STARTED : 4 February 2013
COMPLETED : 4 February 2013

Project No. 19-1351-235

SHEET 1 OF 1
DATUM

Boring method: Solid Stem Augers

DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	COMMENTS
GROUND SURFACE	217.70				
ASPHALT: (100mm)	217.60 0.10				
SAND and **GRAVEL**, very dense, brown, dry: (FILL)		1	SS	61	
		2	SS	34	
CLAY, silty, trace to some sand, trace gravel, stiff to very stiff, brown: (TILL)	216.48 1.22	3	SS	8	
		4	SS	24	Grain Size Analysis: Gr 2%/ Sa 21%/ Si 47%/ Cl 31%
		5	SS	28	
SILT, clayey, trace to some sand, trace gravel, very stiff, reddish grey: (TILL)	213.13 4.57	6	SS	29	
END OF BOREHOLE AT 5.1m. BOREHOLE OPEN TO 5.1m AND DRY. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.3m, CONCRETE TO 0.1m THEN ASPHALT PATCH TO SUREFACE.	212.52 5.18				



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

DEPTH SCALE (metres) 1 2 3 4 5 6 7 8 9

GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION
WATER LEVEL (date)

▼ DEEP/DUAL INSTALLATION
WATER LEVEL (date)

LOGGED : GA
CHECKED : MRA

THURBER

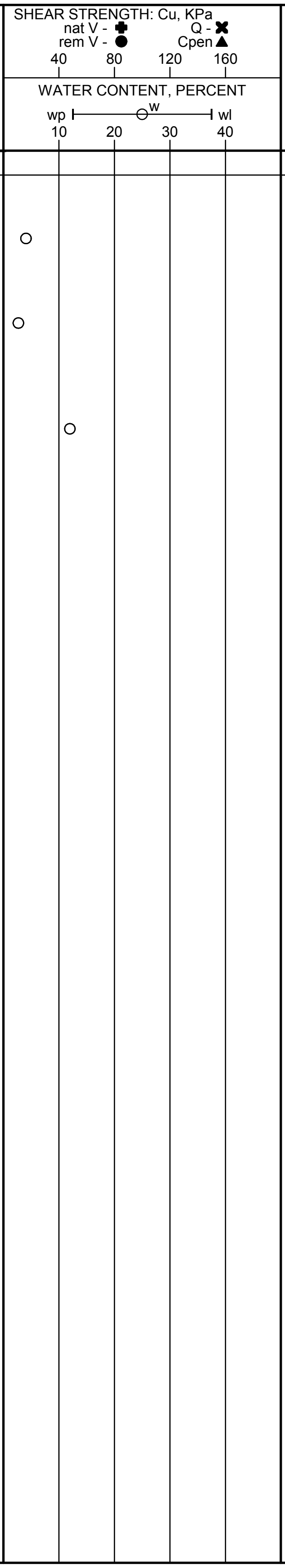
RECORD OF BOREHOLE 13-29

PROJECT : Tremaine Road - Main Street to Steeles Avenue
LOCATION : Milton, ON
STARTED : 4 February 2013
COMPLETED : 4 February 2013

Project No. 19-1351-235
SHEET 1 OF 1
DATUM

DEPTH SCALE (metres)	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	COMMENTS	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	216.80						
	Solid Stem Augers	**ASPHALT:** (100mm)	216.70						
		SAND and **GRAVEL**, some silt, very dense, brown, dry: (FILL)	0.10	1	SS	50	Grain Size Analysis: Gr 17%/Sa 56%/Si & Cl 27%		
1				2	SS	55			
		CLAY, silty, trace to some sand, trace gravel, hard, brown/grey: (TILL)	215.58 1.22	3	SS	34			
2		END OF BOREHOLE AT 2.1m. BOREHOLE OPEN TO 2.1m AND DRY. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.1m THEN ASPHALT PATCH TO SURFACE.	214.67 2.13						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100



Depth scale: 1 2 3 4 5 6 7 8 9

GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION
WATER LEVEL (date)

▼ DEEP/DUAL INSTALLATION
WATER LEVEL (date)

LOGGED : GA
CHECKED : MRA

THURBER

Current Investigation

RECORD OF BOREHOLE 20-01

PROJECT : Steeles Avenue Class EA

LOCATION : Milton, Ontario

STARTED : October 15, 2020

COMPLETED : October 15, 2020

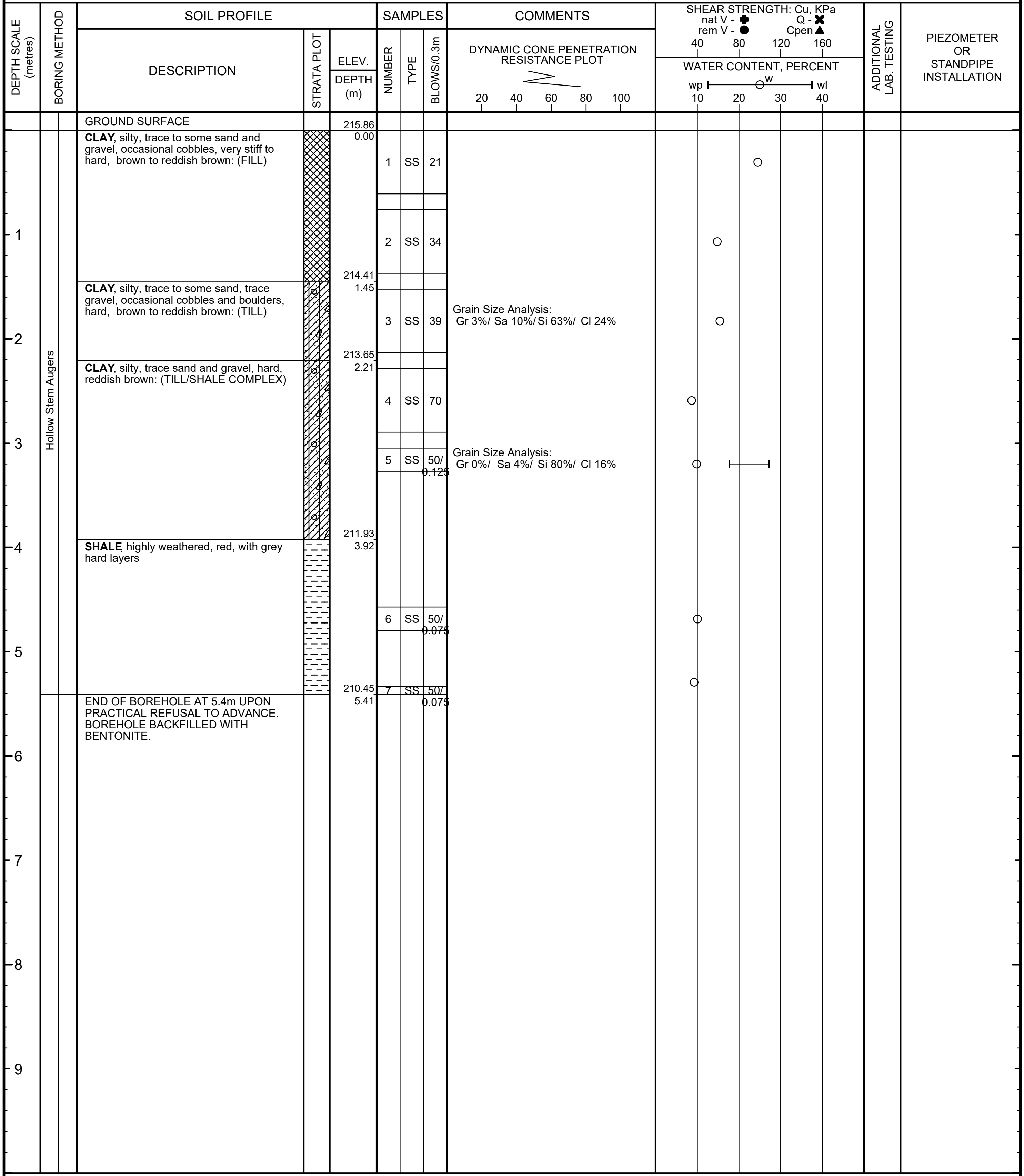
N 4 817 947.5 E 588 738.8

Project No. 29750

SHEET 1 OF 1

DATUM Geodetic

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	COMMENTS
Hollow Stem Augers	GROUND SURFACE	215.86				
	CLAY, silty, trace to some sand and gravel, occasional cobbles, very stiff to hard, brown to reddish brown: (FILL)	0.00	1 2	SS SS	21 34	
	CLAY, silty, trace to some sand, trace gravel, occasional cobbles and boulders, hard, brown to reddish brown: (TILL)	214.41 1.45	3	SS	39	Grain Size Analysis: Gr 3%/ Sa 10%/ Si 63%/ Cl 24%
	CLAY, silty, trace sand and gravel, hard, reddish brown: (TILL/SHALE COMPLEX)	213.65 2.21	4 5	SS SS	70 50/ 0.125	Grain Size Analysis: Gr 0%/ Sa 4%/ Si 80%/ Cl 16%
	SHALE, highly weathered, red, with grey hard layers	211.93 3.92	6 7	SS SS	50/ 0.075 50/ 0.075	
	END OF BOREHOLE AT 5.4m UPON PRACTICAL REFUSAL TO ADVANCE. BOREHOLE BACKFILLED WITH BENTONITE.	210.45 5.41				



GROUNDWATER ELEVATIONS

▽ WATER LEVEL UPON COMPLETION ▼ WATER LEVEL IN WELL/PIEZOMETER

LOGGED : AY

CHECKED : TF

RECORD OF BOREHOLE 20-02

PROJECT : Steeles Avenue Class EA
LOCATION : Milton, Ontario
STARTED : October 15, 2020
COMPLETED : October 15, 2020
N 4 818 144.6 E 588 762.5

Project No. 29750
SHEET 1 OF 1
DATUM Geodetic

BORING METHOD: Hollow Stem Augers

DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	211.35
CLAY, silty, trace sand and gravel, occasional to numerous cobbles and boulders, very stiff to hard, brown to reddish brown: (FILL)	0.00
CLAY, silty, trace sand and gravel, hard, brown to reddish brown: (TILL/SHALE COMPLEX)	209.25 2.10
SHALE, highly weathered, red, with grey hard layers	207.40 3.95
END OF BOREHOLE AT 4.67m UPON PRACTICAL REFUSAL TO ADVANCE. Monitoring Well installation consists of 50mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.	206.67 4.67

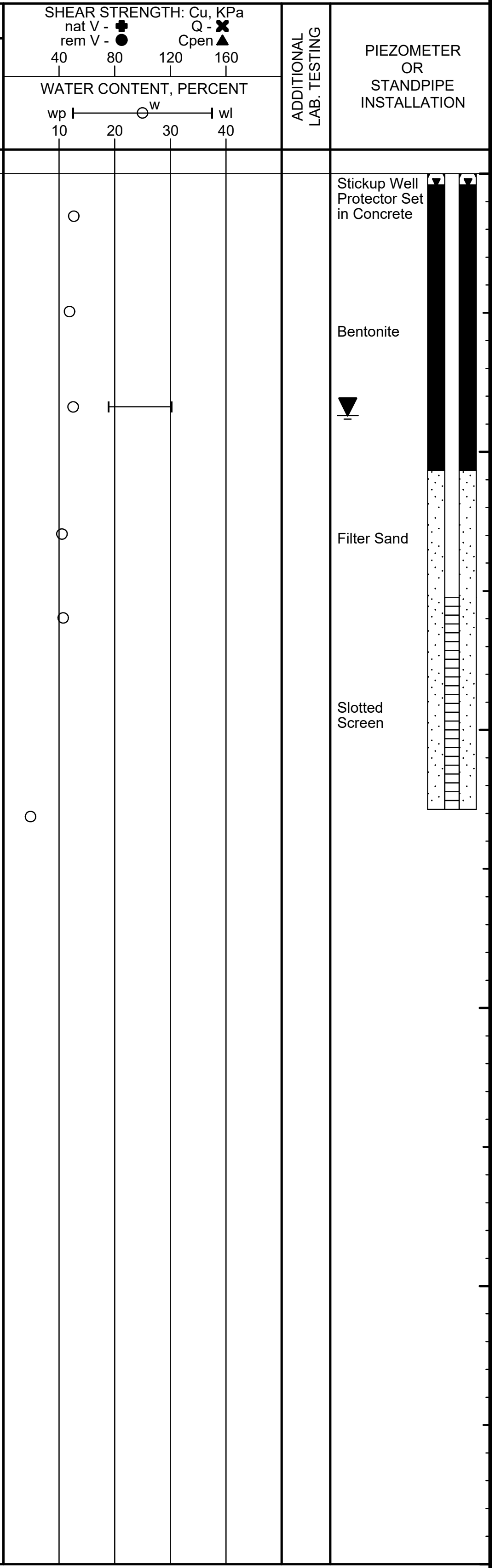
WATER LEVEL READINGS:

DATE	DEPTH(m)	ELEV.(m)
Nov 03/20	1.74	209.61

SAMPLES

NUMBER	TYPE	BLOWS/0.3m	COMMENTS
1	SS	19	
2	SS	50/ 0.125	
3	SS	99/ 0.225	Grain Size Analysis: Gr 0%/ Sa 3%/ Si 76%/ Cl 21%
4	SS	59	Grain Size Analysis: Gr 0%/ Sa 2%/ Si 78%/ Cl 20%
5	SS	50/ 0.125	
6	SS	50/ 0.100	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100



DEPTH SCALE (metres): 1 2 3 4 5 6 7 8 9

GROUNDWATER ELEVATIONS
▽ WATER LEVEL UPON COMPLETION
▼ WATER LEVEL IN WELL/PIEZOMETER November 3, 2020

LOGGED : AY
CHECKED : TF

RECORD OF BOREHOLE 20-03

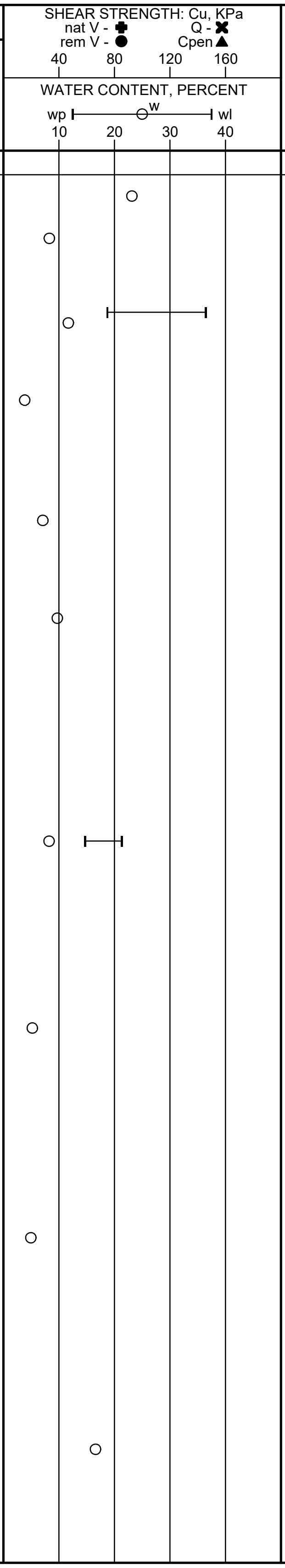
PROJECT : Steeles Avenue Class EA
LOCATION : Milton, Ontario
STARTED : October 14, 2020
COMPLETED : October 14, 2020
N 4 818 413.2 E 588 731.8

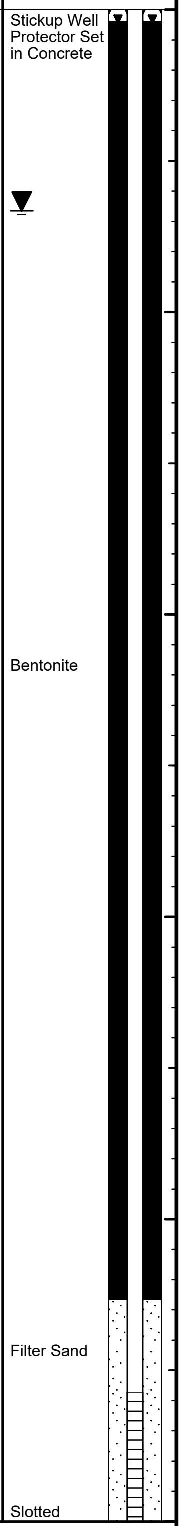
Project No. 29750
SHEET 1 OF 2
DATUM Geodetic

BORING METHOD: Hollow Stem Augers

DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	202.42
TOPSOIL: (150mm)	0.00
CLAY, silty, some sand to sandy, trace gravel, occasional cobbles, occasional roots, very stiff, brown	0.15
CLAY, silty to **SILT**, clayey, some sand, trace gravel, occasional to numerous cobbles, hard, brown: (TILL)	201.02 1.40
Wet	
SHALE, highly weathered, red, with grey hard layers	195.47 6.95

SAMPLES NUMBER	TYPE	BLOWS/0.3m	COMMENTS
1	SS	22	
2	SS	25	Grain Size Analysis: Gr 1%/ Sa 20%/ Si 55%/ Cl 24%
3	SS	50/ 0.075	
4	SS	87/ 0.200	
5	SS	50/ 0.125	
6	SS	82	Grain Size Analysis: Gr 0%/ Sa 15%/ Si 73%/ Cl 12%
7	SS	50/ 0.025	
8	SS	50/ 0.075	
9	SS	50/ 0.075	





DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

DEPTH SCALE (metres): 1 2 3 4 5 6 7 8 9

GROUNDWATER ELEVATIONS

∇ WATER LEVEL UPON COMPLETION
▼ WATER LEVEL IN WELL/PIEZOMETER
November 3, 2020

LOGGED : AY
CHECKED : TF

THURBER

RECORD OF BOREHOLE 20-03

PROJECT : Steeles Avenue Class EA
LOCATION : Milton, Ontario
STARTED : October 14, 2020
COMPLETED : October 14, 2020

N 4 818 413.2 E 588 731.8

Project No. 29750
SHEET 2 OF 2
DATUM Geodetic

DEPTH SCALE (metres)	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	COMMENTS DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	SHEAR STRENGTH: Cu, KPa nat V - ✚ Q - ✖ rem V - ● Cpen ▲ 40 80 120 160 WATER CONTENT, PERCENT wp ⊢ w ○ ⊣ wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
											Screen
		END OF BOREHOLE AT 10.7m UPON PRACTICAL REFUSAL TO ADVANCE. Monitoring Well installation consists of 50mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.		191.68 10.74	10	SS	50/ 0.075		○		
11											
12		WATER LEVEL READINGS:									
13											
14											
15											
16											
17											
18											
19											

WATER LEVEL READINGS:

DATE	DEPTH(m)	ELEV.(m)
Nov 03/20	1.33	201.09

GROUNDWATER ELEVATIONS

∇ WATER LEVEL UPON COMPLETION

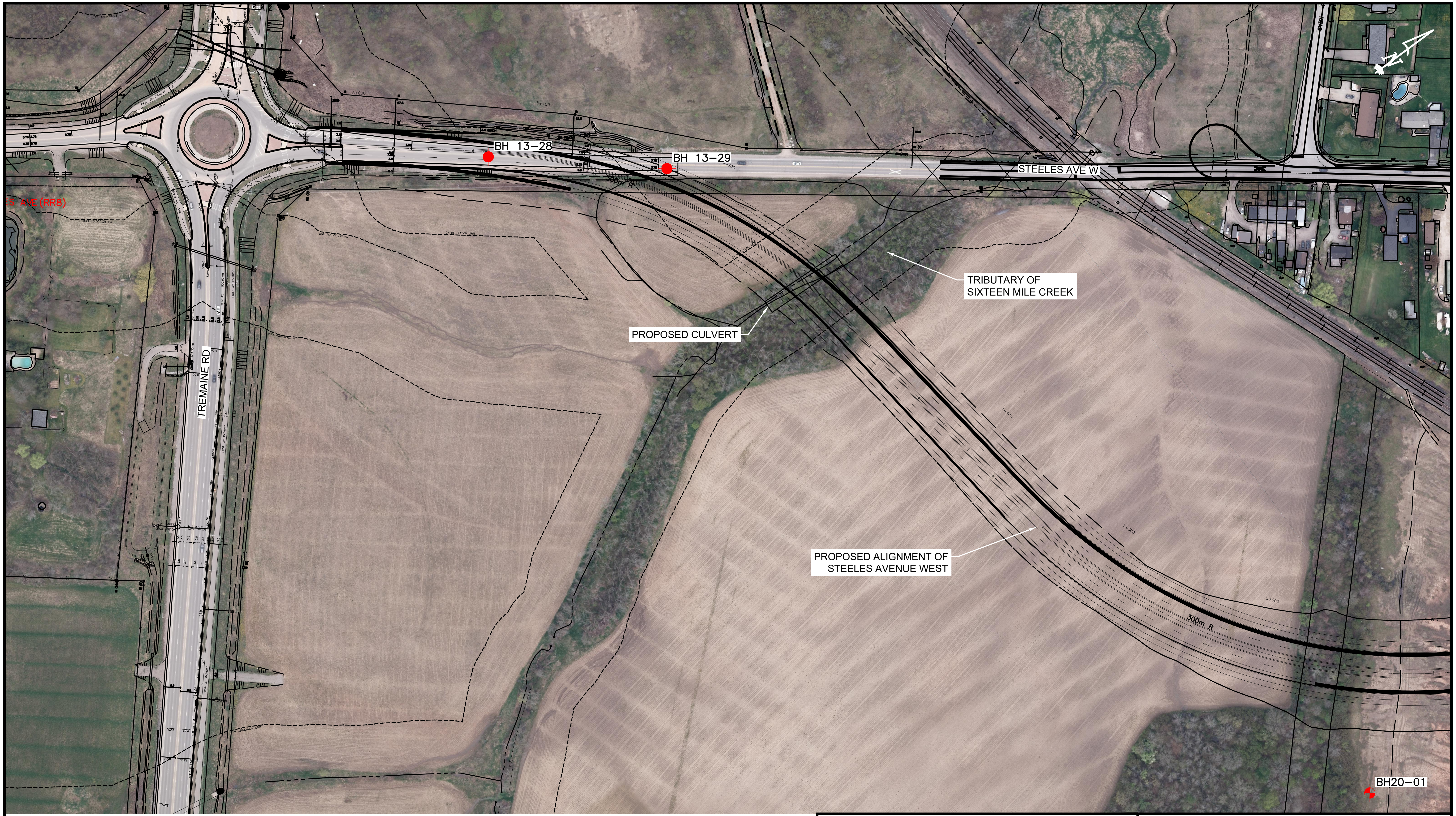
▼ WATER LEVEL IN WELL/PIEZOMETER
November 3, 2020

LOGGED : AY
CHECKED : TF

THURBER

Appendix C

Borehole Location Plans



LEGEND

- ✚ CURRENT INVESTIGATION BOREHOLE(BH) LOCATION
- ● PREVIOUS INVESTIGATION BOREHOLE(BH) LOCATION, THURBER REF.: 19–1351–235
- (MW) MONITORING WELL

0 metres 80

WSP

STEELES AVENUE WEST CLASS EA
FROM TERMAINE RD. TO INDUSTRIAL DR.
MILTON, ONTARIO

BOREHOLE LOCATION PLAN

JOB# 29750

THURBER ENGINEERING LTD.

ENGINEER :	DRAWN :	APPROVED :
TF	BH	RP
DATE :	SCALE :	DRAWING No.
DECEMBER 2020	1:2000	29750-1

FILENAME: H:\Drafting\29000\29750\TED-29750-BHPL.dwg
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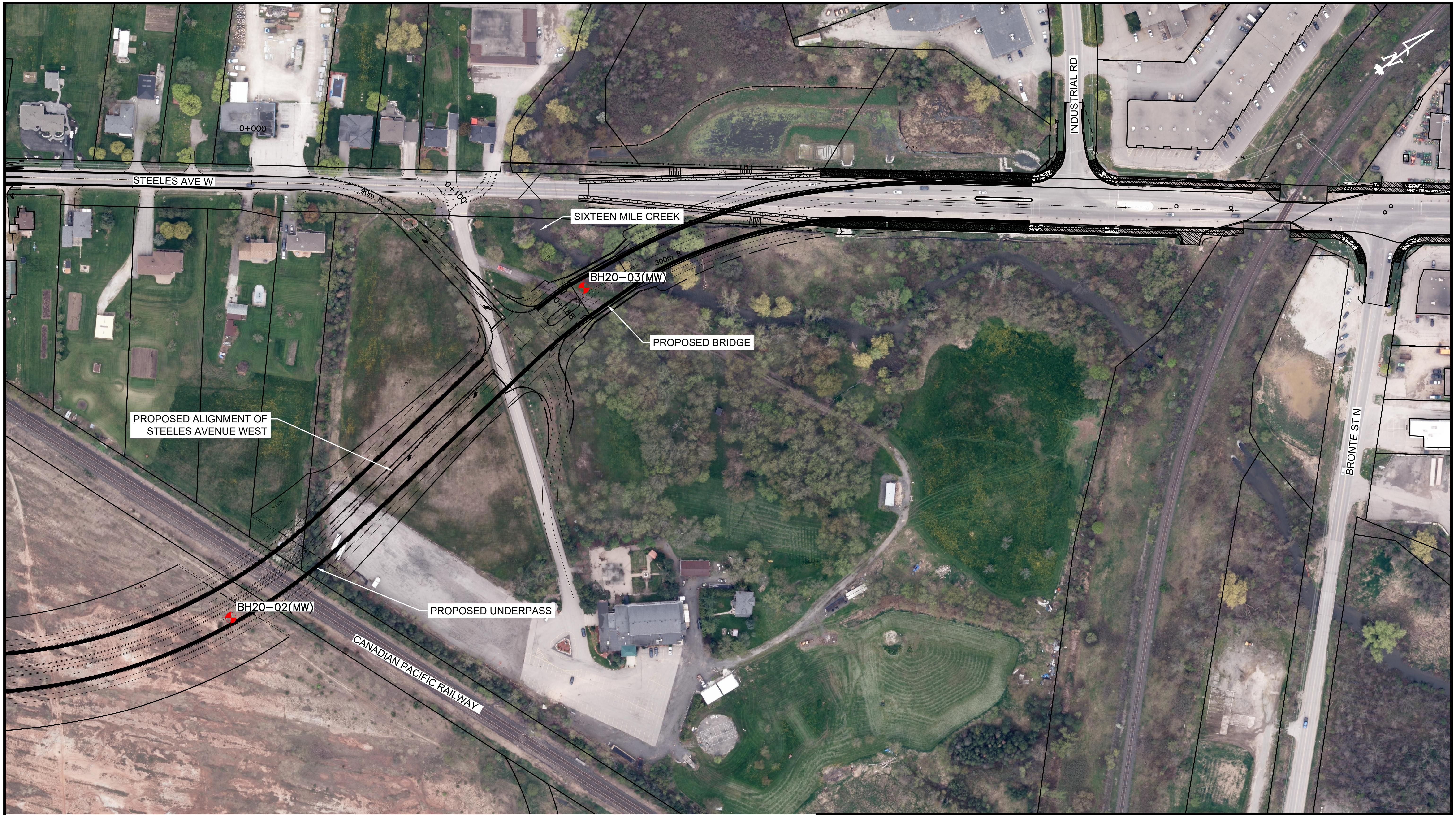


LEGEND

- CURRENT INVESTIGATION BOREHOLE(BH) LOCATION
- ● PREVIOUS INVESTIGATION BOREHOLE(BH) LOCATION, THURBER REF.: 19-1351-235
- (MW) MONITORING WELL

0 metres 80

WSP

STEELES AVENUE WEST CLASS EA
FROM TERMAINE RD. TO INDUSTRIAL DR.
MILTON, ONTARIO

BOREHOLE LOCATION PLAN

JOB# 29750

THURBER ENGINEERING LTD.

ENGINEER : TF	DRAWN : BH	APPROVED : RP
DATE : DECEMBER 2020	SCALE : 1:2000	DRAWING No. 29750-2

FILENAME: H:\Drafting\29000\29750\TED-29750-BHPL.dwg
PLOTDATE: Dec 16, 2020 - 10:58 AM

THURBER

Appendix D

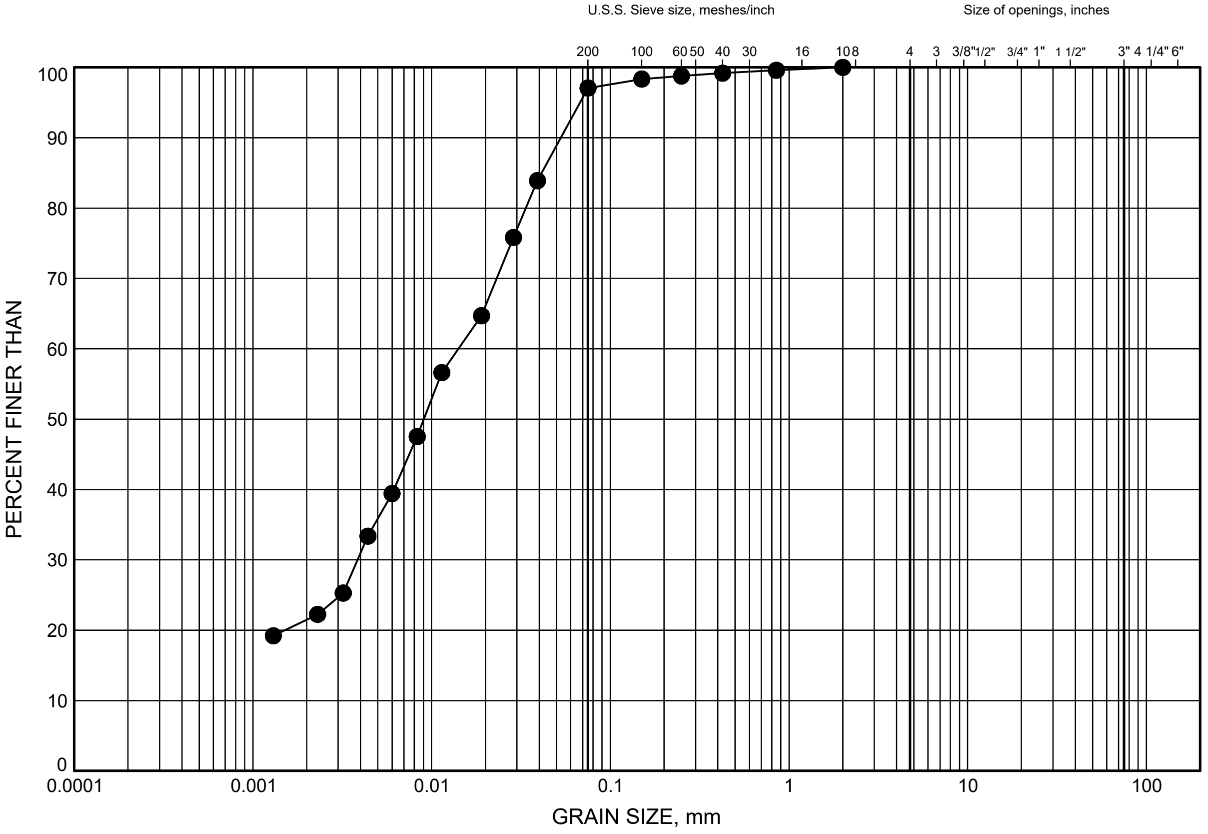
Geotechnical Laboratory Soil Test Results

Steeles Avenue Class EA

GRAIN SIZE DISTRIBUTION

FIGURE D1

Silty CLAY FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	20-02	1.68	209.67

GRAIN SIZE DISTRIBUTION - THURBER TEL-29750.GPJ 12/10/20

Date December 2020

Project 29750

THURBER

Prep'd AN

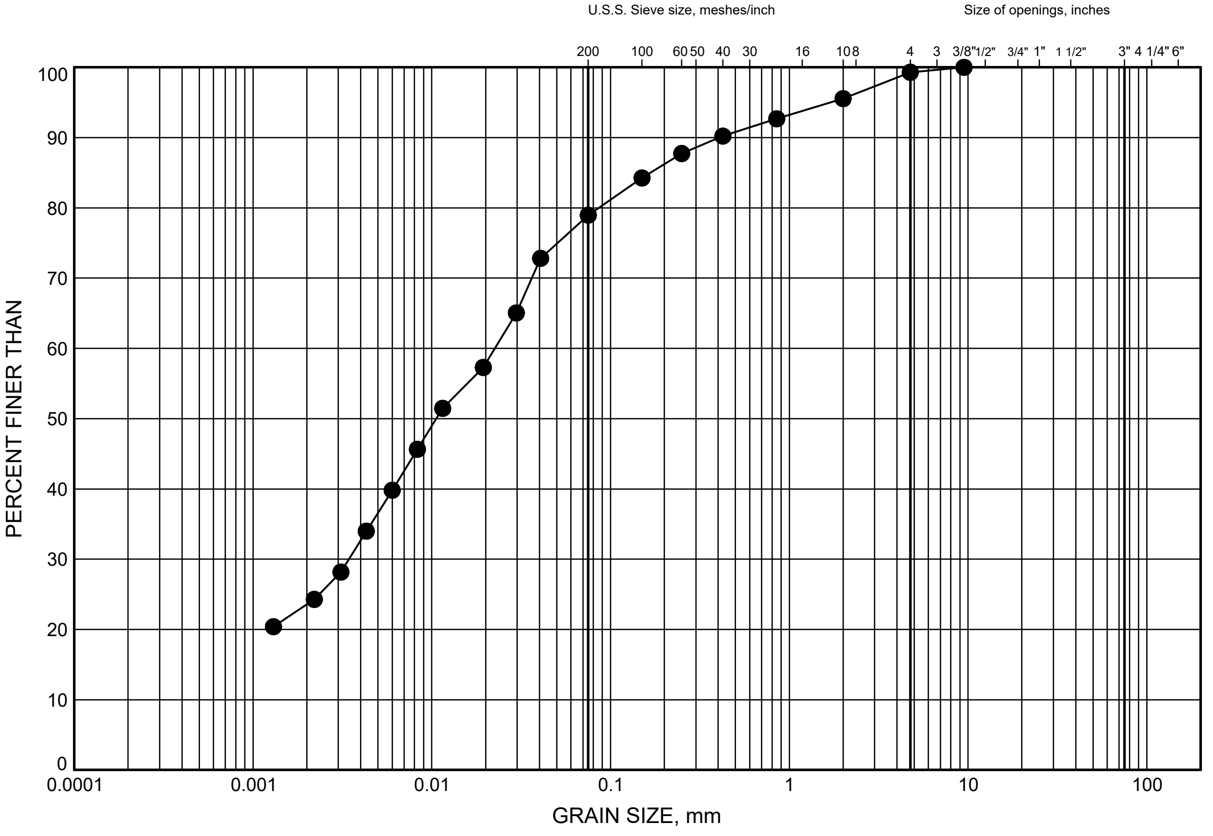
Chkd. TF

Steeles Avenue Class EA

GRAIN SIZE DISTRIBUTION

FIGURE D2

Silty CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	20-03	0.99	201.43

Date December 2020

Project 29750

THURBER

Prep'd AN

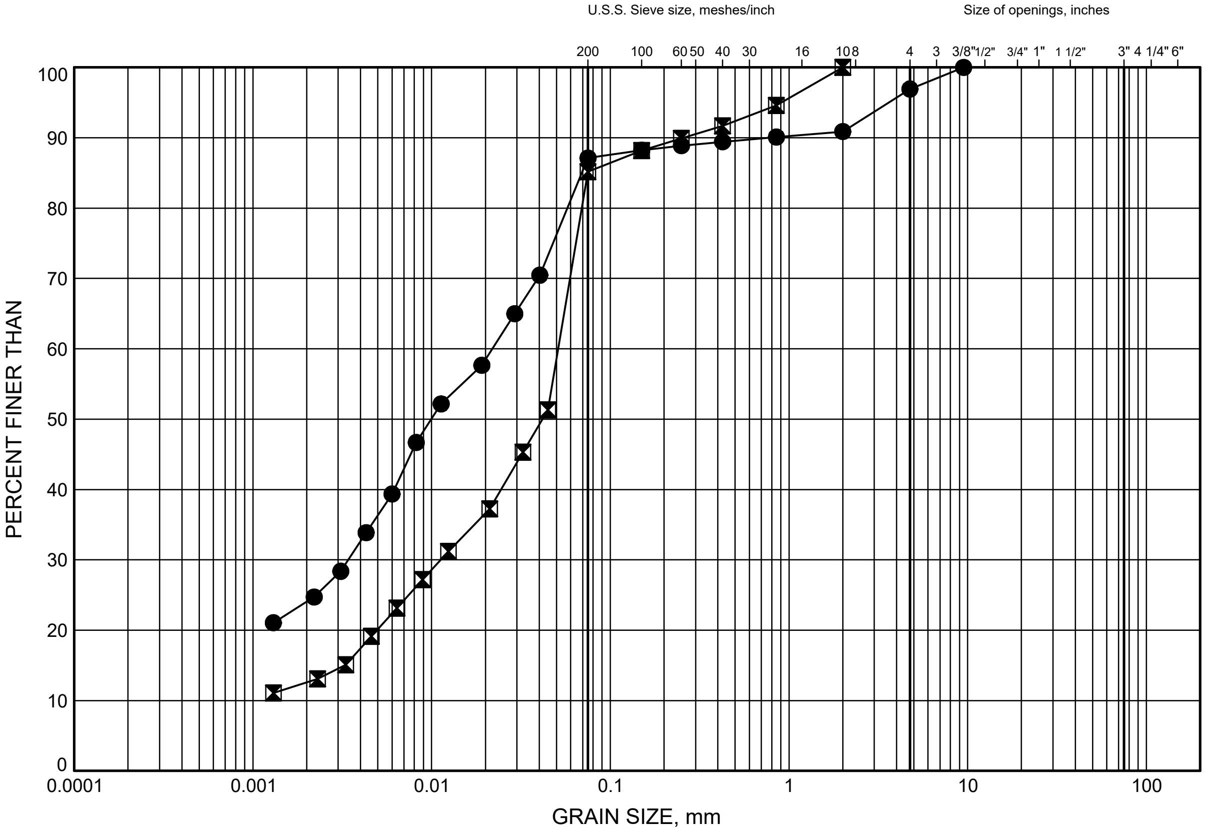
Chkd. TF

Steeles Avenue Class EA

GRAIN SIZE DISTRIBUTION

FIGURE D3

CLAY TILL & SILT TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	20-01	1.83	214.03
⧗	20-03	4.80	197.62

Date December 2020

Project 29750

THURBER

Prep'd AN

Chkd. TF

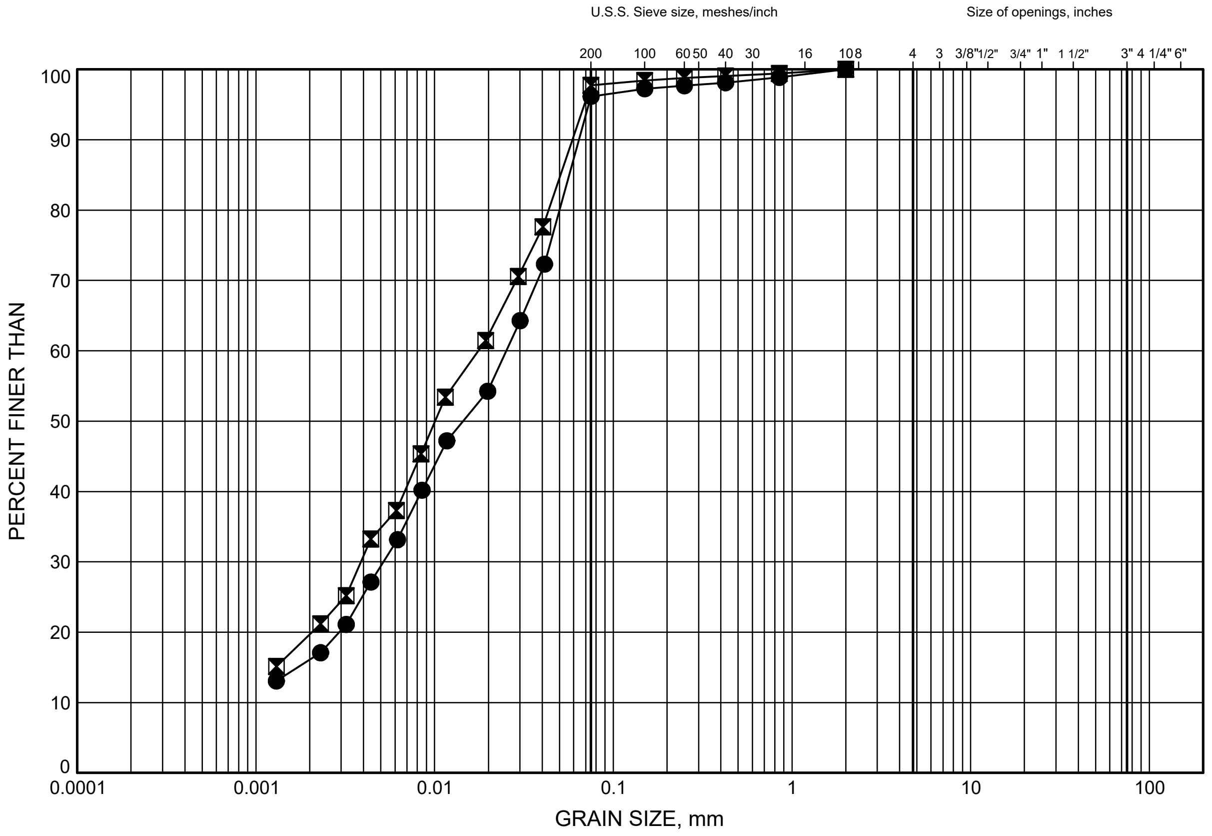
GRAIN SIZE DISTRIBUTION - THURBER TEL-29750.GPJ 12/8/20

Steeles Avenue Class EA

GRAIN SIZE DISTRIBUTION

FIGURE D4

CLAY TILL (Shale Complex)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	20-01	3.20	212.66
☒	20-02	2.59	208.76

Date December 2020

Project 29750

THURBER

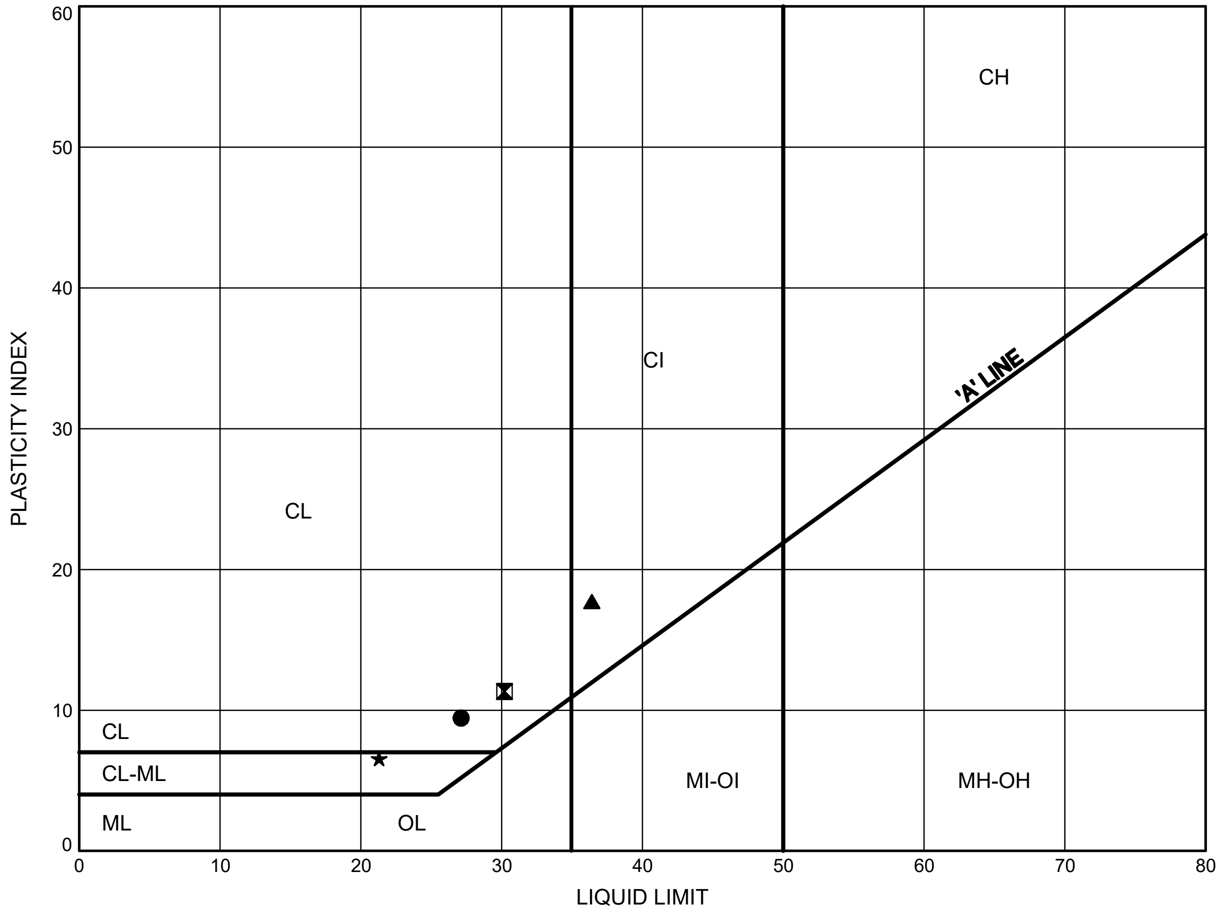
Prep'd AN

Chkd. TF

Steeles Avenue Class EA

ATTERBERG LIMITS TEST RESULTS

FIGURE D5



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	20-01	3.20	212.66
☒	20-02	1.68	209.67
▲	20-03	0.99	201.43
★	20-03	4.80	197.62

Date December 2020

Project 29750

Prep'd AN

Chkd. TF

THURBALT TEL-29750.GPJ 12/8/20