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Appendix I: Geotechnical Investigation Report

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Preliminary Geotechnical Investigation Report

Regional Road 25 (Bronte Road) Transportation Corridor Improvements
Speers Road to Derry Road
Regional Municipality of Halton, Ontario

Client: CIMA+
Date: October 3, 2025
File: 33734

THURBER ENGINEERING LTD.

EXECUTIVE SUMMARY

This report presents the results of a preliminary geotechnical investigation completed for the proposed improvements and potential widening of Regional Road 25 (Bronte Road) between Speers Road in Oakville and Derry Road in Milton in support of the Municipal Class Environmental Assessment (MCEA) Study for the Regional Municipality of Halton.

Halton Regional Road 25 is a major north-south arterial road. The study corridor is approximately 15.7 km in total length and extends in a northwest direction from Speers Road in Oakville to Derry Road in Milton. At present, the roadway primarily consists of a four-lane urban section south of Dundas Street in Oakville and north of Britannia Road in Milton, and a four-lane rural section between Dundas Street and Britannia Road. Major crossings of the corridor include a GO/ Metrolinx rail overpass between Speers Road and Wyecroft Road as well as interchanges at the Queen Elizabeth Way (QEW) and Highway 407 Express Toll Route (ETR).

The geotechnical investigation comprised a review of existing subsurface data from along the corridor and drilling of a total of 36 boreholes to depths of 3.1 to 8.2 m. Monitoring wells were installed in nine boreholes to permit monitoring of the groundwater levels at the site.

The subsurface stratigraphy encountered in the boreholes generally consisted of a pavement structure (asphalt over granular) overlying roadway fill materials, underlain locally by possible alluvial or topsoil layers, overlying native silty clay till. In the deeper boreholes, the till graded to silt and sand with depth, locally overlying shale bedrock. Deposits of clayey silt and sand were encountered at isolated locations.

Based on the results of the investigation, the following preliminary pavement designs are recommended for potential reconstruction and widening of the roadway:

Component	Thickness (mm) Speers Road to QEW	Thickness (mm) QEW to Highway 407	Thickness (mm) Highway 407 to Derry Road
HL1	50	50	50
HDBC (2 lifts)	130	140	130
OPSS Granular A Base	150	150	150
OPSS Granular B Type II Subbase	450	450	450

Pavement design analysis indicates that the existing pavement structure generally meets the structural requirements to carry the estimated 20-year ESALs. Rehabilitation of the existing pavement structure may entail milling 50 mm of the existing asphalt surface followed by replacement with a new 50 mm thick layer of HL1 surface course. The need for an increased overlay thickness and/or crack repair in selected sections should be further assessed during detailed design.

Four bridge structures and nine structural culverts have been identified within the project limits. Preliminary recommendations for widening or extension of the structures are as follows:

- The existing GO/Metrolinx grade separation structure appears to be capable of accommodating six lanes of traffic and therefore widening or replacement is not anticipated. Foundations for retaining walls or other appurtenances, if required for roadway widening, should be supported on spread footings either on shale bedrock or on granular engineered fill placed to establish design founding levels.

- Bronte Road presently comprises six lanes under the QEW overpass structure, consisting of two through lanes in each direction plus a southbound left turn lane to the N/S-E ramp and a northbound auxiliary lane to the S-W ramp. Site observation and the archive general arrangement drawing indicate that the structure was designed to accommodate an additional lane in each direction and therefore widening or replacement is not anticipated. Foundations for retaining walls or other appurtenances, if required for roadway widening, are expected to be founded on shale bedrock.

- At present, Bronte Road primarily comprises six lanes on the Highway 407 underpass structure, consisting of two through lanes in each direction plus a southbound auxiliary lane to the N-E ramp and a northbound auxiliary lane to the S-W ramp. A taper to the left turn lane into a carpool lot also begins in the northbound lanes on the structure. Site observation and the archive general arrangement drawing indicate that the structure was not designed to accommodate additional lanes and therefore bridge widening will likely be required.

  To a provide similar a foundation performance as the existing structure, the preferred foundation system for structure widening will consist of steel H-piles at the abutments and extension of the spread footing at the pier. Based on existing information, the abutment piles will be driven to refusal in shale bedrock and the abutment footing extension will be founded on weathered shale bedrock.

- The existing Sixteen Mile Creek bridge carries four lanes of traffic and a multi-use trail on the east side. Widening of the bridge will be required to accommodate six lanes. To provide similar foundation performance as the existing structure, the preferred foundation system for the widening will consist of steel H-piles. Considering the Sixteen Mile Creek parallels the west side of the roadway north of the bridge, the widening is expected to be on the east side of the structure, in close proximity to an existing watermain. To mitigate potential disturbance of the watermain, the piles should be socketed into shale in accordance with the design previously employed.

- In general, the subsurface stratigraphy at the culvert locations consists of a pavement structure and roadway embankment fill underlain by probable alluvial deposits, underlain by stiff to hard silty clay till. For extension or replacement of box and pipe culverts, preparation of the culvert subgrade prior to placement of the bedding material should include subexcavation of the fill and organic/alluvial deposits. Footings for extension or replacement of open-footing or arch culverts should be founded on native undisturbed soil below the level of the fill and organic/alluvial materials, generally at the same level as the existing culvert footings.

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APPENDICES

1. INTRODUCTION

Thurber Engineering Ltd. (Thurber) was retained by CIMA+ to conduct a preliminary geotechnical investigation in support of the Municipal Class Environmental Assessment (MCEA) Study for the Regional Municipality of Halton for improvements to the Regional Road 25 transportation corridor between Speers Road in Oakville and Derry Road in Milton (RPF # P-1065-21). The anticipated works to be completed as part of the infrastructure improvements include widening of the roadway to six lanes, active transportation facilities, priority transit, and development of vertical and horizontal alignments including new structures where required.

The purpose of the investigation was to review existing geotechnical information along the corridor, further explore the subsurface conditions 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 foundation design for structure widenings, culvert foundations, pavement design, subgrade preparation, backfilling, earthworks, and slope stability if applicable.

The scope of work for this preliminary assessment did not include an environmental analytical testing program to evaluate the environmental quality of excess soils that may be generated during the proposed construction works. In addition, the work scope did not include a hydrogeological site assessment or sampling to address Ontario Ministry of the Environmental, Conservation and Parks (MECP) EASR registration or Permit to Take Water (PTTW) application.

Thurber Engineering Ltd. (Thurber) carried out the investigation as a sub-consultant to CIMA+ who is conducting the MCEA Study for the Regional Municipality of Halton.

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 and Project Description

The project corridor is approximately 15.7 km in total length and extends in a northwest direction from Speers Road in Oakville to Derry Road in Milton. For the purposes of this report, the corridor is considered to extend south to north. At present, the roadway primarily consists of a four-lane urban section south of Dundas Street in Oakville and north of Britannia Road in Milton, and a four-lane rural section between Dundas Street and Britannia Road. Major crossings of the corridor include a GO/Metrolinx rail overpass between Speers Road and Wyecroft Road as well as interchanges at the Queen Elizabeth Way (QEW) and Highway 407 Express Toll Route (ETR). The project corridor is shown on Figure 33734-1, appended.

The southern section of the corridor from Speers Road to Dundas Street is situated in an urban setting primarily comprising commercial and industrial properties from Speers Road to 1.0 km north of the QEW, and residential subdivisions with localized retail and institutional properties north of Upper Middle Road. Bronte Creek meanders north to south within a deeply incised valley to the west of Bronte Road from approximately 1.0 km north of the QEW to the south project limit, and Fourteen Mile Creek passes west to east under Bronte Road approximately 200 m north of Upper Middle Road.

The section from Dundas Street to Britannia Road is located within a rural setting consisting of farmland and residential dwellings along with occasional commercial and institutional properties, including a golf course on the east side of Bronte Road north of Lower Base Line.

The section from Britannia Road to Derry Road comprises a mix of urban and rural land use, with most of the lands having recently been developed or currently undergoing development for residential and community use. Sixteen Mile Creek flows north to south adjacent to the west side of Bronte Road from the north project limit before crossing the road approximately 300 m north of Louis St. Laurent Avenue.

In general, the topography along the corridor is gently undulating. The ground surface along the alignment generally rises towards the north, from near Elev. 100 m at the intersection of Regional Road 25 and Speers Road, to approximate Elev. 194 m at the Derry Road intersection. Topographic contours along the project corridor are shown on Figure 33734-2, appended.

2.2 Geology

Based on the information in *The Physiography of Southern Ontario*[1] by Chapman and Putnam (1984), the site is located within three physiographic regions as shown on Figure 33734-2, appended, and further described below:

- The Iroquois Plain, from the south limit to approximately 1.1 km north of Speers Road. The Iroquois Plain was formed in the late Pleistocene times by a body of water known as Lake Iroquois, which emptied eastward at Rome, New York. Lake Iroquois was

[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.

characterized by higher water levels than the present-day Lake Ontario, caused by an ice sheet blocking the present-day St. Lawrence River valley. When the St. Lawrence valley became free of ice, the water level dropped to a level much lower than the present Lake Ontario levels (*Pleistocene Geology of the Hamilton Map-Area*[2]).

- The South Slope, from approximately 1.1 km north of Speers Road to 600 m north of Henderson Road. The South Slope is characterized by low-lying, fine-grained, undulating ground moraine and knolls.
- The Peel Plain, from approximately 600m north of Henderson Road to the north project limit. 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.

Based on *Surficial Geology of Southern Ontario*[3], the surficial deposits along the south part of the corridor (south from 600 m north of Henderson Road) generally comprise clay to silt textured till derived from glaciolacustrine deposits or shale. Along the north part, the surficial soils primarily consist of fine-textured glaciolacustrine deposits of silt and clay with minor sand and gravel. Shallow Paleozoic bedrock is indicated at the QEW interchange and a coarse-textured glaciolacustrine deposit comprising sand, gravel with minor silt and clay extends north of this area to Saw Whet Boulevard. Modern alluvial deposits consisting of clay, silt, sand, gravel with possible organic matter are identified along Sixteen Mile Creek between Louis St. Laurent Avenue and Derry Road. The surficial geology mapping is presented on Figure 33734-4, appended.

According to *Paleozoic Geology of Southern Ontario*[4], the bedrock underlying the corridor consists of Queenston Formation shale with minor limestone, siltstone or sandstone. The bedrock geology mapping is shown on Figure 33734-5, appended.

2.3 Previous Investigations

A number of geotechnical investigations have been carried out previously along the corridor. The following sections provide a summary of the available geotechnical reports and drawings reviewed in preparation of this report. Selected geotechnical data is provided in Appendix A.

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[2] Karrow, P. F., 1959; Pleistocene Geology of the Hamilton Map-Area. Ontario; Toronto, Ontario. Ontario Department of Mines

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

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

Reference is made to the reports for additional details, including investigation procedures, laboratory testing results and detailed subsurface conditions, which are not provided herein.

2.3.1 GO/Metrolinx - Bronte Road Grade Separation

A geotechnical investigation was completed in 1984 for a proposed two-span grade separation structure over Bronte Road adjacent to the north side of the existing GO/Metrolinx structure between Speers Road and Wyecroft Road. The findings were documented in the following report: MTO GEOCRES No. 30M5-286, "Geotechnical Investigation, GO-ALRT Programme, Oakville Project, West Extension, W.O. 82-26025, Bronte Road Subway, District 4, Hamilton", Peto MacCallum Ltd, March 1984.

The field investigation consisted of three boreholes, one at each abutment and one at the pier, advanced to depths of 4.1 to 8.9 m. In general, the subsurface stratigraphy encountered in the boreholes comprised a pavement structure or silty clay fill overlying shale bedrock at depths of 0.7 to 3.4 m (Elev. 97.9 to 103.1 m). The shale was described as red with occasional greyish green layers and poor to very poor quality, becoming fair to excellent quality at depths of 2.9 to 8.1 m (Elev. 95.7 to 99.9 m). Water was not observed in the boreholes prior to rock coring.

2.3.2 Wyecroft Road Extension

Thurber completed a geotechnical investigation during the period August 2020 to July 2021 for the proposed Wyecroft Road Extension from Burloak Drive to east of Bronte Road, currently under construction. The factual information obtained during the investigation was documented in a report titled "Geotechnical Data Report, Wyecroft Road Extension, Burloak Drive to East of Bronte Road, Oakville, Ontario" and dated February 8, 2022.

The field investigation included three boreholes drilled on Bronte Road (P-06 to P-08) as well as five boreholes (P-05, P-09, SWM2-02, TUN-07 and TUN-08) located within approximately 50 m of Bronte Road. In general, the subsurface stratigraphy encountered in the boreholes consisted of a pavement structure or topsoil layer overlying stiff to hard silty clay till, underlain by shale bedrock contacted at depths of 0.5 to 1.7 m (Elev. 101.2 to 106.8 m). The pavement structure encountered on Bronte Road consisted of 225 to 250 mm of asphalt overlying 250 to 425 mm of granular material.

In general, the boreholes were open and dry upon completion and prior to bedrock coring. Groundwater levels measured in monitoring wells installed in two of the boreholes ranged from 4.2 to 5.8 m depth (Elev. 100.4 to 101.2 m) below the ground surface. An additional monitoring well was dry to 4.6 m depth (Elev. 101.4 m).

2.3.3 Bronte Road - QEW Overpass

Thurber carried out foundation investigations for MTO in 2002 to 2006 for widening of the QEW from Third Line to Burloak Drive, including replacement of the Bronte Road overpass structure. The investigations included the following boreholes from within approximately 50 m of existing Bronte Road:

- Boreholes 06-1 to 06-16 from "Foundation Investigation and Design Report, Bronte Road Overpass, QEW/Bronte Road Interchange, Oakville, Ontario G.W.P. 169-00-00, Site No. 10-586" dated January 31, 2007.

- Boreholes BR1 to BR5, EN4, NW1, NW2, NWC1 to NWC4 and SW1 from "Foundation Investigation and Design Report, Bronte Road Deep Cuts, QEW Widening, Third Line to Burloak, G.W.P. 169-00-00" dated February 9, 2007.

- Boreholes NW1, NW2, NWC2 and NWC3 were also included in "Foundation Investigation and Design Report, Retaining Walls and Privacy Wall, QEW Widening, Third Line to Burloak Drive, G.W.P. 169-00-00" dated January 18, 2007.

- Boreholes SWM1 and UC7 from "Foundation Investigation and Design Report, Culvert, SWM Pond, Utility Crossings and Watermain Protection, QEW Widening, Third Line to Burloak Drive, G.W.P. 169-00-00" dated January 22, 2007.

In general, the subsurface conditions encountered in the boreholes consisted of a surficial topsoil layer or QEW pavement structure overlying relatively thin, discontinuous deposits of fill, sand, silty clay and silty clay till underlain by reddish brown shale bedrock contacted at depths ranging from 0.7 to 5.8 m (Elev. 108.1 to 117.9 m). The shale was typically described as highly weathered in the upper 1 to 2 m and becoming moderately to slightly weathered with depth.

Groundwater was observed in five boreholes at depths of 1.1 to 5.2 m (Elev. 110.8 to 120.9 m) upon completion of drilling. Piezometers were installed in selected boreholes (Boreholes 06-1, 06-13, BR2, BR4, BR5, NW2, NWC1 to NWC3, and SWM1) to measure water levels after drilling. Groundwater was measured in the piezometers at depths of 1.9 to 11.3 m (Elev. 105.5 to 120.1 m).

2.3.4 Watermain and Sewer Extension, Bronte Road and Upper Middle Road

Thurber carried out a geotechnical investigation in November 2004 for installation of a 1200 mm diameter watermain under Fourteen Mile Creek and extension of a 2400 mm diameter sewer near the southeast quadrant of the Bronte Road and Upper Middle Road intersection ("Geotechnical

Investigation, Proposed Watermain and Sewer Extension, Bronte Road and Upper Middle Road, Town of Oakville, Halton Region", dated July 15, 2005). The field investigation included two boreholes (Boreholes 04-3WM and 04-4WM) located within 50 m of the west side of Bronte Road, north of the Fourteen Mile Creek crossing, and one borehole (Borehole 04-5WM) drilled on Upper Middle Road approximately 50 m east of Bronte Road.

The subsurface stratigraphy encountered in the boreholes generally consisted of a pavement structure or fill layer underlain by native very stiff to hard clayey silt till, grading to dense to very dense silt and sand till at depths of 3.8 to 5.2 m. Very dense gravelly sand with cobbles was encountered below the till at depths of 5.2 and 8.2 m (Elev. 120.6 and 122.0 m), to the borehole termination depths of 9.1 and 9.8 m.

Groundwater was measured in a piezometer installed adjacent to Fourteen Mile Creek at 3.4 m depth (Elev.122.4 m).

2.3.5 Realignment of Bronte Road

Thurber completed a geotechnical investigation for the realignment of Bronte Road between Upper Middle Road and Hwy 407 in 2004 ("Geotechnical Investigation Report, Reconstruction of Bronte Road (Regional Road 25), Upper Middle Road to Hwy 407, Town of Oakville, Halton Region" dated September 16, 2004). The widening project extended from approximately 500 m south of Upper Middle Road to the south limit of the Highway 407 interchange. The scope of the geotechnical investigation consisted of 78 boreholes drilled between November 2003 and January 2004, ranging in depth from 1.2 to 11.0 m. Two additional boreholes were drilled in May 2005 in association with a proposed retaining wall.

In general, the subsurface stratigraphy encountered in the boreholes drilled along the existing and proposed Bronte Road alignment consisted of a pavement structure, fill, topsoil layer, or "reworked till" overlying predominant silty clay till. The pavement structure encountered on existing Bronte Road comprised 50 to 390 mm of asphalt (average 225 mm) overlying granular material to a total depth of 610 to 750 mm. Where encountered, the maximum depth of fill was 4.6 m. Below the upper softened zone at the ground surface, the silty clay till was very stiff to hard and graded to clayey silt till at depth and/or locally. Highly weathered, reddish brown shale bedrock was contacted in 15 boreholes at depths ranging from 1.4 to 4.6 m, 9.0 m at one location.

In general, the boreholes were reported as "dry" upon completion of drilling. Water levels were observed at the ground surface to 1.4 m depth in three boreholes. Groundwater levels measured in piezometers installed in three boreholes ranged from 1.0 to 3.9 m below ground surface.

The pavement structure design recommended for widening, new construction or reconstruction of Bronte Road comprised 140 mm of asphalt overlying 150 mm of Granular A (19 mm crushed limestone) and 600 mm of Granular B Type II. The recommended rehabilitation treatment for the existing pavement involved milling and placement of a 100 to 140 mm thick asphalt overlay.

2.3.6 Highway 25 – Highway 407 Underpass

A foundation investigation was completed in 1990 for the underpass structure carrying Highway 25 (Regional Road 25) over Highway 403 (now Highway 407). The findings were documented in the following report: MTO GEOCRES No. 30M5-167, "Foundation Investigation Report for Bridge Structure, Hwy. 403 – Hwy. 25 Underpass, W.P. 409-85-02, Site No. 10-479, District 4, Burlington", dated December 1990.

The field investigation consisted of seven boreholes drilled to depths of 4.5 to 6.2 m in April and May 1990. In general, the subsurface stratigraphy encountered in the boreholes consisted of sand and gravel fill, clayey silt fill and/or clayey silt topsoil overlying native clayey silt till, underlain by Queenston Shale bedrock. The surficial fill and topsoil layers extended to depths of 0.8 to 1.7 m (Elev. 163.7 to 165.5 m). The consistency of the till was stiff to hard. Bedrock was encountered at depths ranging from 1.9 to 3.1 m (Elev. 162.6 to 164.2 m).

Groundwater was encountered at 4.4 m depth (Elev. 160.8 m) in one borehole during the investigation and measured at depths of 3.0 and 3.8 m (Elev. 163.3 and 162.8 m) in two other boreholes. The remaining boreholes were reported to be dry upon completion of drilling.

2.3.7 Sixteen Mile Creek Bridge

A foundation investigation was completed in 1969 for replacement of the bridge carrying Highway 25 (Regional Road 25) over Boyne River (Oakville Creek, now Sixteen Mile Creek) between Louis St. Laurent Avenue and Derry Road. The findings were documented in the following report: MTO GEOCRES No. 30M5-079, "Foundation Investigation Report for The proposed Boyne River Bridge on Hwy. 25. Town of Oakville, District No. 4, W.J. 69-F-54, W.P. 132-65", dated September 1969.

The field investigation consisted of four boreholes drilled to depths of 8.4 to 12.2 m in July 1969. The subsurface stratigraphy encountered in the boreholes consisted of a topsoil layer overlying glacial till comprising clayey silt, sand and gravel, underlain by shale bedrock. The till was generally described as hard and very dense. Bedrock was contacted at 7.8 m depth (Elev. 174.2 to 174.4 m) in three boreholes and was not contacted at the borehole termination depth of 12.2 m (Elev. 173.7 m) in the other borehole.

The reported groundwater level upon completion of drilling was at the ground surface (Elev. 181.9 to 182.1 m) in three boreholes and at 2.7 m depth (Elev. 183.2 m) in the remaining borehole. In one borehole, an artesian condition was encountered above the bedrock surface at approximately 7.0 m depth (Elev. 175.0 m), with the water level rising to 1.5 m above the ground surface (Elev. 183.5 m).

2.3.8 Regional Road 25 Structure Widenings – Britannia Road to Derry Road

Thurber completed a geotechnical investigation in 2007 for widening of existing structures along Regional Road 25 between Britannia Road and Derry Road as part of a previous widening project. The proposed works included widening of the existing Sixteen Mile Creek bridge as well as extension or replacement of five culverts.

The field investigation consisted of fifteen boreholes advanced to depths ranging from 6.1 to 16.7 m and six hand probes to depths of 0.6 to 0.9 m at the various structure locations. In general, the subsurface stratigraphy encountered in the boreholes and hand probes consisted of surficial topsoil, granular material and/or clay fill overlying a discontinuous silty clay layer overlying hard clayey silt to silty clay till. Where encountered, the fill extended to depths of 2.1 to 7.0 m. The till graded to sandy silt to sand and silt till in five boreholes and was locally overlain by a sand and gravel deposit. Shale bedrock was contacted below the till at depths of 5.5 to 12.2 m (Elev. 174.6 to 177.3 m) in three boreholes at the Sixteen Mile Creek bridge and at 4.3 m depth (Elev. 181.1 m) in a culvert borehole south of Derry Road.

Groundwater was observed in two boreholes at depths of 1.7 and 3.7 m upon completion of drilling. Piezometers were installed in five boreholes as part of the investigation; groundwater was measured in the piezometers at depths ranging from 0.2 to 2.6 m (Elev. 181.1 to 185.2 m).

2.3.9 Highway 25 Sewer and Watermain Project

A geotechnical investigation was completed in 1999 for installation of a 500 to 900 mm diameter trunk watermain and an 825 to 1350 mm diameter trunk sanitary sewer along Regional Road 25 from Upper Middle Road in Oakville to Derry Road/Thompson Road in Milton. The findings were documented in the following report: “Geotechnical Design Report, Highway 25 Sewer and Watermain Project, Region of Halton, Ontario”, PR-1823 and PR-2186A, Terraprobe Limited, dated February 1999.

The field investigation consisted of 44 boreholes drilled along the alignment in December 1998 and January 1999. In general, the boreholes were drilled at approximate 500 m intervals to depths of 6 to 15 m. Within a proposed tunnel/forcemain area extending from just north of Lower Baseline

Road to just north of Burnhamthorpe Road, the boreholes were advanced at 200 m intervals to depths of 15 to 25 m.

The subsurface stratigraphy encountered in the boreholes generally comprised a layer of fill material overlying native clayey silt till underlain by a discontinuous layer of sands and silts and shale bedrock. The fill extended to depths of up to 4.0 m with an average depth of approximately 1.4 m. The consistency of the till was typically very stiff to hard, locally stiff. The discontinuous deposits of sands and silts were generally less than 3 m thick, described as water-bearing, and reported in 25% of the boreholes. The underlying shale was contacted at depths of 1.4 to 21.5 m in most boreholes located south of Britannia Road, and at 10.0 m depth at the Sixteen Mile Creek crossing. The bedrock surface generally rises from Elev. 126.2 at Upper Middle Road to Elev. 167.3 m at Burnhamthorpe Road and varies from about Elev. 167 m to Elev. 173 m north of Burnhamthorpe Road. The bedrock is described as red weathered shale with dolostone layers, with an upper weathered portion becoming progressively less weathered with depth.

In general, groundwater observations were not indicated on the borehole logs. Piezometers were installed in nine boreholes. Groundwater levels were measured at depths of 5.0 to 11.8 m in three piezometers upon completion of drilling and at depths of 1.4 to 11.8 m in six piezometers in February 1999. Five piezometers could not be located after installation.

3. INVESTIGATION PROCEDURES

3.1 Borehole Drilling

The field investigation for this project was carried out from December 4 to 14, 2023 and comprised a total of thirty-six (36) boreholes; twenty-eight (28) of which were targeted to assess pavement structures (P-01 to P-28) and eight (8) targeted to culvert assessments (C-01 to C-08) as summarized below:

Table 3.1: Borehole Details

Facility	Borehole No.	Approx. Ground Elevation (m)	Borehole Termination Depth (m)	Approx. Borehole Termination Elevation (m)
Culverts	C-01 to C-08	129.3 to 187.9	7.6 to 8.2	121.7 to 179.6
Pavement Structure, Municipal Services	P-01 to P-28	100.7 to 193.8	3.1 to 8.2	94.4 to 190.2

The approximate locations of the boreholes are shown on Drawings 33734-01 to 33734-10 provided in Appendix B. Borehole Details are provided in the Record of Borehole sheets included in Appendix C.

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 R12i GNSS receiver. The ground surface elevations are orthometric derived from GNSS observations and Natural Resources Canada Geoid Model HT2.0, which is a height transformation model that was created by modifying a gravimetric geoid model (CGG2000) to fit the historical CGVD28:78 datum. Borehole location coordinates are presented in the Universal Transverse Mercator (UTM) system (UTM-17 NAD83).

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.

Boreholes were advanced using solid stem augers powered by a truck mounted CME 55 drill rig supplied and operated by DBW 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 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 for further examination and testing.

3.2 Monitoring Well Details

Groundwater conditions were observed in the open boreholes throughout the drilling operations. Monitoring wells were installed in Boreholes C-01 to C-08 and P-16 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 dept within the borehole. The wells were provided with flush mounted surface access covers. The installation details are summarized in Table 3.2 below.

Table 3.2: Monitoring Well Details

Borehole/ Monitoring Well No.	Ground Surface Elevation(m)	Monitoring Well Tip Depth (m)	Monitoring Well Tip Elevation (m)	Slotted Screen Length (m)	Mid-Screen Depth (m)	Mid-Screen Elevation m)
C-01	129.3	7.6	121.7	1.5	6.8	122.5
C-02	130.1	7.6	122.5	1.5	6.8	123.3
C-03	166.4	6.7	159.7	1.5	5.9	160.5
C-04	174.6	7.4	167.2	1.5	6.6	168.0
C-05	177.7	7.5	170.2	1.5	6.7	171.0
C-06	179.5	7.5	172.0	1.5	6.7	172.8
C-07	184.1	7.5	176.6	1.5	6.7	177.4
C-08	187.9	7.6	180.3	1.5	6.8	181.1

P-16	191.6	3.0	188.6	1.5	2.2	189.4

The boreholes in which no monitoring wells were installed were backfilled in general accordance with Ontario Regulation 903.

4. LABORATORY TESTING

Geotechnical laboratory testing was carried out at Thurber's laboratory. All recovered soil samples were subjected to visual inspection 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. Results of the geotechnical soil laboratory testing are presented on the Record of Borehole sheets in Appendix C and in detail in Appendix D.

5. DESCRIPTION OF SUBSURFACE CONDITIONS

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

The subsurface stratigraphy encountered in the boreholes generally consisted of a pavement structure (asphalt over granular) overlying roadway fill materials, underlain locally by possible alluvial or topsoil layers, overlying native silty clay till. In the deeper boreholes, the till graded to silt and sand with depth, locally overlying shale bedrock. Deposits of clayey silt and of sand were encountered at isolated locations.

5.1 Pavement Structure

All boreholes were drilled through the pavement structure on Bronte Road. The pavement structure typically consisted of 150 to 375 mm of asphalt (average 240 mm) overlying 815 to 2010 mm (average 1060 mm) of granular base materials. A buried layer or fragment of asphalt was encountered within the granular material in Boreholes C-06, C-08, and C-26. In Borehole P-01, the granular material was 355 mm thick and underlain by silt fill containing an asphalt layer/ fragment at 1.0 m depth.

SPT N-values recorded in the granular material ranged from 9 blows per 0.3 m of penetration to 50 blows per 0.025 m of penetration, indicating typically very dense to compact relative density, decreasing with depth. Measured moisture contents ranged from 1% to 14%, typically less than 5%.

The results of grain size distribution analyses carried out on samples of the granular fill materials are presented on Figures D1 and D2 in Appendix D. The results of the grain size distribution analyses are summarized in the table below.

Soil Particle	Percentage (%)
Gravel	21 to 43
Sand	38 to 55
Silt and Clay	14 to 26

The samples tested do not meet the OPSS Granular A or B gradation specifications due to the percentage of silt and clay sized particles. The results may be impacted by the effects of compaction, auger sampling procedures, infiltration of fines with road runoff, or deterioration of the granular material over time.

5.2 Fill

A layer of fill was encountered below the pavement structure in Boreholes C-01 to C-08, P-03, P-05, P-06, P-13, P-16 and P-27 at depths of 1.1 m to 1.5 m (Elev. 128.7 m to 190.3 m). The fill was penetrated at depths of 2.0 m to 4.1 m (Elev. 126.0 m to 189.4 m). The fill deposit typically consisted of cohesive silty clay, locally cohesionless silty sand to silt. In Borehole P-01, the fill was encountered at 0.5 m depth (Elev. 100.1 m) and consisted of reddish brown silt to 1.2 m depth, underlain by silty clay to 3.0 m depth, and silt and sand to the borehole termination depth of 6.3 m (Elev. 94.4 m).

SPT N-values recorded in the fill generally ranged from 4 to 30 blows per 0.3 m of penetration, indicating a firm to very stiff consistency or loose to compact relative density. Localized N-values of 85 blows per 0.3 m and 60 blows per 0.18 m of penetration (very dense) were recorded in Boreholes C-03 and P-01, respectively. Measured moisture contents ranged from 2% to 23%.

The results of grain size distribution analyses carried out on samples of the fill materials are presented on Figure D3 to D5 in Appendix D. The results of the grain size distribution analyses are summarized in the table below.

Soil Particle	Percentage (%)
Gravel	0 to 10
Sand	18 to 41
Silt	33 to 64
Clay	11 to 18

Atterberg limit analyses of a sample of the fill measured a liquid limit, plastic limit, and plasticity index of 22, 18 and 4, respectively. These results, which are plotted on Figure D10 in Appendix D, indicate that the sample tested consists of slightly plastic silt (CL-ML).

5.3 Topsoil and Alluvial Deposits

Possible alluvial material was encountered below the pavement structure and fill in Boreholes C-01, C-03, C-05, C-06, P-05, P-16, P-17, P-19 and P-23. In addition, a topsoil layer was encountered below the fill in Boreholes P-06, P-26 and P-27. The topsoil and alluvial deposits were encountered at depths of 1.2 m to 3.5 m (Elev. 126.4 m to 192.1 m) and typically varied from 0.2 m to 1.9 m in thickness. The organic deposits were generally described as brown to black silty clay. Locally in Borehole P-05, a 3.1 m thick deposit of sand and gravel alluvium was encountered, underlain by silty clay alluvium to the borehole termination depth of 8.2 m (Elev. 124.0 m).

SPT N-values of 5 to 13 blows per 0.3 m of penetration was recorded in the organic deposits, indicating a firm to stiff consistency, locally a loose to compact relative density. Measured moisture contents of 11% to 26% were determined, with one value of 4% in the sand and gravel.

5.4 Sand

A layer of sand, some silt, was encountered below the fill in Borehole P-03 at 2.7 m depth (Elev. 127.3 m) and was penetrated at 3.7 m depth (Elev. 126.3 m). An SPT N-value of 6 blows per 0.3 m of penetration was recorded in the sand, indicating a loose relative density. A moisture content of 14% was measured.

5.5 Clayey Silt

Clayey silt was encountered below the pavement structure in Boreholes P-02 and P-04 and below the sand layer in Borehole P-03 at depths of 1.3 m to 3.7 m (Elev. 124.6 m to 128.8 m). The clayey silt layer was 1.0 m and 2.3 m thick in Boreholes P-01 and P-04, respectively with thicknesses varying from 0.7 m to 2.3 m. Borehole P-03 was terminated in this layer at a depth of 4.4 m (Elev. 125.6 m).

SPT N-values of 9 to 24 blows per 0.3 m of penetration were recorded in this layer, indicating a stiff to very stiff consistency. Moisture contents of 12% to 22% were determined.

The results of a grain size distribution analysis carried out on a sample of the clayey silt are presented on Figure D6 in Appendix D. The results indicated 0% gravel, 1% sand, 70% silt and 29% clay sized particles.

Atterberg limit analysis of a sample of the clayey silt measured a liquid limit, plastic limit, and plasticity index of 26, 20 and 6, respectively. These results, which are plotted on Figure D11 in Appendix D, indicate that the sample tested consists of slightly plastic silt (CL-ML).

5.6 Silty Clay Till

Silty clay till was encountered below the pavement structure, fill and alluvial/topsoil layers in all boreholes except Boreholes C-08, P-01, P-03 and P-05. The native clay till was encountered at depths of 1.1 m to 4.9 m (Elev. 123.6 m to 192.4 m). In Boreholes C-01 to C-07 and P-02, the clay till was penetrated at depths of 3.3 m to 7.0 m (Elev. 122.7 m to 180.0 m). Boreholes P-04 and P-06 to P-28 were terminated in the silty clay till layer at depths of 3.1 m to 4.4 m (Elev. 125.6 m to 190.2 m).

SPT N-values recorded in the clay till typically ranged from 19 to 78 blows per 0.3 m of penetration, with localized values of up to 50 blows per 0.05 m of penetration, indicating a very stiff to hard consistency. At seven locations, lower N-values of 7 to 14 blows per 0.3 m were obtained near the upper boundary of this unit, indicating a firm to stiff consistency. Measured moisture contents ranged from 4% to 23%, typically about 10% to 19%.

The results of grain size distribution analyses carried out on samples of the silty clay till are presented on Figures D7 and D8 in Appendix D. The results of the grain size distribution analyses are summarized in the table below.

Soil Particle	Percentage (%)
Gravel	0 to 5
Sand	15 to 26
Silt	46 to 54
Clay	19 to 33

Atterberg limit analyses were completed on selected samples of the silty clay till. The results of the testing are presented in Figure D12 in Appendix D and in the table below.

Soil Particle	Percentage (%)
Liquid Limit	24 to 35
Plastic Limit	16 to 20
Plasticity Index	8 to 16

According to the Atterberg results, the tests indicate that the samples tested consist of low plastic clay (CL).

Though not encountered in the boreholes, glacially derived till soils often contain cobbles and boulders, and these should be anticipated when excavating during construction.

5.7 Silt to Silt and Sand (Till to Till/Shale Complex)

A deposit of silt to silt and sand till was encountered below the silty clay till in Boreholes C-01, C-02, C-05 to C-08, and P-02 at depths of 3.3 m to 7.0 m (Elev. 122.7 m to 183.7 m). This till contained trace to some gravel and trace to some clay. All boreholes which encountered the silt to silt and sand till were terminated in this layer at depths ranging from 3.5 m to 8.2 m (Elev. 121.7 m to 179.6 m).

A silt till/shale complex was encountered below the silty clay till in Boreholes C-03 and C-04 at 5.6 m depth (Elev. 160.9 m and 169.0 m, respectively). The silt till/shale complex was described as reddish brown and containing some sand (to sandy), some clay, trace gravel, and shale fragments. Both boreholes penetrated the till/shale complex at 6.9 m depth (Elev. 159.5 m and 167.7 m).

SPT N-values recorded in the till to till/shale complex ranged from 48 blows per 0.3 m of penetration to 50 blows for 0.025 m of penetration, indicating a typically very dense relative density. Measured moisture contents ranged from 3% to 15%.

The results of grain size distribution analyses carried out on samples in this layer are presented on Figure D9 in Appendix D. The results of the grain size distribution analyses are summarized in the table below.

Soil Particle	Percentage (%)
Gravel	3 to 14
Sand	23 to 39
Silt	39 to 60
Clay	8 to 17

Atterberg limit analysis of two selected samples measured liquid limits, plastic limits, and plasticity indices of 20 and 23, 16, and 4 and 7, respectively. These results, which are plotted on Figure D13 in Appendix D, indicate that the sample tested consists of slightly plastic silt (CL-ML).

5.8 Shale Bedrock

Bedrock was encountered underlying the overburden soils in Boreholes C-03 and C-04. The depths and elevations at which the bedrock was encountered are summarized in Table 5.1 below.

Table 5.1: Bedrock Contact Depths and Elevations

Borehole	Bedrock Surface Depth (m)	Bedrock Surface Elevation (m)
C-03	6.9	159.5
C-04	6.9	167.7

Bedrock was determined by auguring and SPT sampling in the upper zone of the highly weathered shale. SPT N-values recorded in the bedrock were 50 blows per 0.02 m and 0.05 m of penetration. Bedrock coring was not completed during this investigation.

In general, the bedrock was described to be highly weathered, red shale.

5.9 Groundwater Observations

Upon completion of drilling, groundwater was observed at depths of 1.2 m to 5.5 m (Elev. 96.0 m to 182.2 m) in Boreholes P-01, P-03, P-05, P-21, P-22, and P-27. In addition, cave was noted at depths of 7.3 m and 1.2 m (Elev. 124.9 and 182.1) in Boreholes P-05 and P-22 respectively. The remaining boreholes without monitoring wells were open and dry.

Monitoring wells were installed in Boreholes C-01 to C-08 and P-16 to monitor groundwater levels. The measured water levels are summarized in Table 5.2 below.

Table 5.2: Groundwater Measurements

Borehole/ Monitoring Well No.	Ground Surface Elev. (m)	Mid-Screen Depth (m)	Mid-Screen Elev. (m)	Screened Soil Unit	Ground Water Level (m) Ground Water Elevation (m) December 20, 2023
C-01	129.3	6.8	122.5	Silt Till	6.8 122.6
C-02	130.1	6.8	123.3	Clay Till to Silt Till	7.2 122.9
C-03	166.4	5.9	160.5	Clay Till to Silt Till	6.6 159.8
C-04	174.6	6.6	168.0	Silt Till to Shale	2.9 171.7
C-05	177.7	6.7	171.0	Silt Till	3.7 174.0
C-06	179.5	6.7	172.8	Silt Till	3.4 176.1

C-07	184.1	6.7	177.4	Silt and Sand Till	2.5 181.7
C-08	187.9	6.8	181.1	Silt and Sand Till	3.3 184.6
P-16	191.6	2.2	189.4	Clay Fill and Alluvium	DRY

The groundwater levels are short-term readings and fluctuations of the groundwater levels are to be expected. In particular, the groundwater levels may be at a higher elevation after periods of significant or prolonged precipitation.

6. ENGINEERING DISCUSSION AND RECOMMENDATIONS

This section of the report presents interpretation of the data obtained during previous and current subsurface investigations and presents comments and preliminary recommendations for pavement rehabilitation and widening as well as potential modification or replacement of the bridges and culverts along the study area.

The comments and recommendations presented in this report are based on the subsurface soil and groundwater conditions encountered during the investigations and on the preliminary design concept available at the time of this report. In this regard, the comments and recommendations are considered preliminary and should be reviewed and/or revised, if required, during the detailed design stage. Subsurface conditions may vary between and beyond the borehole locations.

6.1 Pavement Design and Construction

6.1.1 Design Analysis

Regional Road 25 is a major north-south arterial road extending between the Towns of Oakville and Milton in Halton Region. At present, the roadway primarily consists of a four-lane urban section south of Dundas Street in Oakville and north of Britannia Road in Milton, and a four-lane rural section between Dundas Street and Britannia Road.

The posted speed limit varies from 60 km/h between Speers Road and Dundas Street West, 70 km/h between Dundas Street West and 700 m north of William Halton Parkway, 80 km/h between 700 m north of William Halton Parkway and 400 m south of Britannia Road, 70 km/h between 400 m south of Britannia Road and 300 m south of Derry Road, and 50 km/h from 300 m south of Derry Road to Derry Road.

Potential improvements include widening of the road to a six-lane urban cross-section with either six general purpose lanes or four general purpose lanes plus two transit only lanes or transit only/ high occupancy vehicle (HOV) lanes.

Mid-block traffic data (calculated AADT) for 2022 was provided by CIMA+. The traffic information is summarized in Table 6.1, delegated into five segments with relatively similar traffic volumes. The average truck volume was 3% and 5% for AM and PM periods. An annual growth rate of 2% was indicated for preliminary design.

Table 6.1: Regional Road 25 Traffic Information

Segment	AADT (2022)
Speers Road to QEW	23,480 to 34,320
QEW to Upper Middle Road	45,620 to 51,500
Upper Middle Road to Highway 407	34,470 to 42,650
Highway 407 to Britannia Road	26,710 to 34,080
Britannia Road to Derry Road	24,940 to 27,910

The traffic data was used to determine the pavement damage caused by 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 2.15 and a construction completion date of 2029, the number of ESALs during a 20-year design period was computed to range from 8.6 to 13.5 million in the various sections of Regional Road 25.

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) = 0.47
- Mean soil resilient modulus (MR) = 30 MPa

The subgrade for the pavement structure is expected to consist primarily of discontinuous layers of fill and silty clay overlying very stiff to hard native silty clay till.

Based on the design input parameters and calculated ESALs, design structural numbers (SN_{Des}) ranging from 141 to 148 mm are required for the various sections. The recommended pavement design thickness, based on the structural requirements, traffic projections, and subgrade conditions, are presented below.

The traffic assessment and structural requirements for pavement design and rehabilitation must be confirmed when additional traffic data is available, particularly in areas of potentially higher truck volumes (ie., Speers Road to QEW) and in transit lanes.

6.1.2 Recommended Pavement Design

Based on the borehole data, the selected input values, calculated ESALs, and design SN_{Des} , the following preliminary pavement designs are recommended for widening and/or full reconstruction of Regional Road 25:

Table 6.2: Pavement Design for Regional Road 25

Component	Thickness (mm) Speers Road to QEW	Thickness (mm) QEW to Highway 407	Thickness (mm) Highway 407 to Derry Road
HL1	50	50	50
HDBC (2 lifts)	130	140	130
OPSS Granular A Base	150	150	150
OPSS Granular B Type II Subbase	450	450	450

It is noted that the average total depth of the existing pavement structure encountered in the boreholes was approximately 1.3 m. For the purposes of preliminary design and cost evaluation, the granular subbase thickness in widening areas should be increased to 850 mm to approximately match the frost penetration and existing total pavement depths to maintain lateral drainage of the existing granular materials.

Pavement design analysis indicates that the existing pavement structure generally meets the structural requirements to carry the estimated 20-year ESALs. Rehabilitation of the existing pavement structure may entail milling 50 mm of the existing asphalt surface followed by replacement with a new 50 mm thick layer of HL1 surface course.

Based on visual observation of the existing pavement surface, several sections may require additional rehabilitation such as a thicker overlay, crack repair prior to overlay, or full depth asphalt replacement. Sections warranting further investigation and assessment in this regard include the following:

- Speers Road to Wyecroft Road where slight to moderate map cracking is evident throughout.
- Dundas Street to Highway 407 where slight to moderate centreline cracking is extensive between lanes, intermittent transverse cracks are present, and a few areas of wheel-track cracking are noted.
- Britannia Road to Derry Road, between areas of intersection improvements, where extensive slight map and longitudinal cracking is evident and has largely been sealed. Much of the cracking is present within the middle two lanes, possibly indicating that resurfacing has been carried out in sections of the outer lanes and/or cracking has reflected through an overlay of a previous two-lane pavement.

Full depth asphalt replacement should be considered for areas of pavement that will be significantly cut by construction activities such as installation of storm sewers and other underground utilities, replacement of culverts, shifting of lane alignments, or relocation of traffic islands, raised medians and curbs. Further, partial depth asphalt replacement may not be cost-effective or efficient to construct in areas requiring profile or grade changes.

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 maximum 20-year design ESALs for Regional Road 25 was estimated to be 13.5 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.MUNI 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.

6.1.3 Pavement Subgrade Preparation

Pavement subgrade preparation should include removal of surficial vegetation, topsoil, organic or compressible material. Grading to the new top of subgrade should match or exceed the thickness of the existing pavement to maintain lateral drainage at the top of subgrade. The exposed subgrade should be compacted and proof-rolled with a heavy roller and examined 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.MUNI 804 and 805.

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

6.2 Bridge Structures

Based on review of OSIM reports, archive geotechnical borehole plans, and site observations, four bridge structures have been identified along the Regional Road 25 corridor within the project limits. The structures are summarized below in Table 6.3.

Table 6.3: Bridge Locations

Structure	Type	Year of Construction	Referenced Boreholes*
GO/Metrolinx Grade Separation	Four span solid slab concrete, 4x13.3m spans, 18.3 m wide	1980	84-1, 84-2, 84-3
Bronte Road - QEW Overpass	EBL – single span CPCI girder, 39.3 m span, 31.7 m wide WBL – single span CPCI girder, 39.1 m span, 29.6 m wide	2012	06-01 to 06-16
Bronte Road - Highway 407 Underpass	Two span prestressed concrete girder, 38m+38m span, 32 m wide	2000	90-1 to 90-7
Sixteen Mile Creek Bridge	Single span concrete girder, 30 m span, 22.45 m wide	2013	69-1 to 69-4, 07-05 to 07-08

**Borehole numbers from previous investigations are prefixed by the year of drilling (i.e., 90-6 indicates Borehole 6 drilled in 1990)*

Geotechnical comments regarding foundations for bridge and approach widening, if required, are provided in the following sections.

6.2.1 GO/Metrolinx Grade Separation

Bronte Road currently comprises four lanes under the existing four-span grade separation structure, with two lanes passing under the middle spans in each direction. The bridge appears to have been designed to allow for widening to six lanes and therefore bridge widening or replacement is not anticipated.

Details regarding the bridge foundations were not available at the time of this report. Based on the existing Geocres information, shale bedrock is present at shallow depth, and it is inferred that the structure is supported on spread footings founded on bedrock. Foundations for retaining walls or other appurtenances, if required for roadway widening to six lanes, should also be supported on spread footings, founded either on shale bedrock or on granular engineered fill placed to establish design founding levels. For preliminary design purposes, the following factored geotechnical resistances may be assumed for footings constructed on shale or engineered fill:

Table 6.4: Recommended Spread Footing Design

Founding Stratum	Factored Geotechnical Resistance (kPa) ULS	Factored Geotechnical Resistance (kPa) SLS
Shale Bedrock	1,000	DNG
Engineered Fill	600	350

DNG = does not govern

The profile drawing provided in the Geocres report indicates that the grade separation was constructed by fill placement in the approaches and excavation into the bedrock. The elevation of the existing bedrock surface encountered in the boreholes therefore varies, from Elev. 103.1 and 101.3 m at the east and west abutments to Elev. 97.9 m at the centre pier in the road cut. Subsurface investigation would be required to establish bedrock elevations and recommended founding levels within the footprint of proposed structure foundations.

Where founding levels of adjacent footings differ, the founding elevation between footings should be stepped in maximum 0.6 m steps at a maximum inclination of 10 horizontal to 7 vertical.

Engineered fill, if required, should consist of OPSS Granular A material placed in maximum 200 mm thick lifts and compacted to 100% of the Standard Proctor Maximum Dry Density (SPMDD), within 2% of the optimum moisture content. The granular pad should extend laterally at least 1 m from the top of footing then outwards and downwards at an inclination no steeper than 1H:1V.

The resistance values are for footings 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 (2019) Clauses 6.10.2 to 6.10.4.

The lateral resistance developed along the base of concrete footings founded on shale or engineered fill may be computed using ultimate friction coefficients of 0.5 and 0.6, respectively. A geotechnical resistance factor of 0.8 should be applied to these ultimate values.

6.2.2 QEW Overpass Structure

Bronte Road presently comprises six lanes under the QEW overpass structure, consisting of two through lanes in each direction plus a southbound left turn lane to the N/S-E ramp and a northbound auxiliary lane to the S-W ramp. Site observation and the archive general arrangement drawing indicate that the structure was designed to accommodate an additional lane in each direction and therefore widening or replacement is not anticipated.

Based on the information presented in the existing Foundation Investigation Report, the Bronte Road cut has been excavated some 4 to 8 m into shale bedrock. Foundations for retaining walls or other appurtenances, if required for roadway widening to six lanes, are expected to be founded on shale bedrock. For preliminary purposes, spread footings constructed on shale bedrock may be designed using a factored geotechnical resistance of 1,000 kPa at ULS. The SLS resistance will not govern design.

Where founding levels of adjacent footings differ, the founding elevation between footings should be stepped in maximum 0.6 m steps at a maximum inclination of 10 horizontal to 7 vertical.

The resistance values are for footings 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 (2019) Clauses 6.10.2 to 6.10.4.

The lateral resistance developed along the base of concrete footings founded on shale may be computed using an ultimate friction coefficient of 0.5. A geotechnical resistance factor of 0.8 should be applied to these ultimate values.

6.2.3 Highway 407 Underpass Structure

At present, Bronte Road primarily comprises six lanes on the Highway 407 underpass structure, consisting of two through lanes in each direction plus a southbound auxiliary lane to the N-E ramp and a northbound auxiliary lane to the S-W ramp. A taper to the left turn lane into a carpool lot also begins in the northbound lanes on the structure. Site observation and the archive general arrangement drawing indicate that the structure was not designed to accommodate additional lanes and therefore bridge widening will likely be required.

The archive general arrangement drawing for the Highway 407 underpass indicates that the existing structure consists of two spans of 38.0 m length and has a total width of approximately 32.0 m. The abutments were designed as integral abutments supported on HP 310x110 piles driven to design tip elevations of 161.0 m. A factored axial capacity at ULS of 1600 kN/pile is specified. The pier was designed to be supported on a 3.7 m wide, 33.9 m long, and 1.1 m thick spread footing with a top of footing at Elev. 165.75 m. The geotechnical capacity used for footing design was not shown.

To a provide similar a foundation performance as the existing structure, the preferred foundation system for the bridge widening will consist of steel H-piles at the abutments and extension of the spread footing at the pier. Based on the Geocres information, the abutment piles will be driven to refusal in shale bedrock and the abutment footing extension will be founded on weathered shale bedrock.

A factored geotechnical resistance of 1,600 kN at ULS may be employed for preliminary design of HP310x110 piles. The SLS resistance is not expected to govern design of the pile foundations driven into bedrock.

It is noted that the pile at each end of the abutments was installed on a 1:10 batter. New piles for widenings must be spaced at least three pile widths from the toe of the existing piles to avoid contacting the existing piles during driving.

To reduce resistance to lateral movement and provide a relatively flexible pile system for integral abutment design, the top of each pile should be installed in a pre-augered hole supported by a CSP and filled with loose sand as per MTO Structural Office Report SO-96-01.

Factored geotechnical resistances of 750 kPa at ULS and 500 kPa at SLS may be employed for preliminary design of spread footings founded on the weathered bedrock at the design founding level of Elev. 164.65 m. The resistance at SLS is provided for a settlement not exceeding 25 mm. Differential settlement is expected to be less than 75% of this value.

Where founding levels of adjacent footings differ, the founding elevation between footings should be stepped in maximum 0.6 m steps at a maximum inclination of 10 horizontal to 7 vertical.

The resistance values are for footings 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 (2019) Clauses 6.10.2 to 6.10.4.

The lateral resistance developed along the base of concrete footings founded on shale may be computed using an ultimate friction coefficient of 0.5. A geotechnical resistance factor of 0.8 should be applied to these ultimate values.

6.2.4 Sixteen Mile Creek Bridge

The existing Sixteen Mile Creek bridge carries four lanes of traffic and a multi-use trail on the east side. Widening of the bridge will therefore be required to accommodate six lanes.

The archive general arrangement drawing indicates that the existing bridge is an integral abutment structure supported on HP 310x110 piles designed using factored axial capacities of 1600 kN/pile and 1400 kN/pile at ULS and SLS, respectively. To mitigate vibration and disturbance to an existing 900 mm diameter watermain, the piles on the east side of the bridge were to be installed in 600 mm diameter pre-augered holes socketed 3 m into shale, to a tip elevation of 171.5 m. The remaining piles were to be driven to achieve the design resistance in very dense/hard till at an estimated pile tip elevation of 177.0 m.

To provide similar foundation performance as the existing structure, the preferred foundation system for the widening will consist of steel H-piles. Considering the Sixteen Mile Creek parallels the west side of the roadway north of the bridge, the widening is expected to be constructed on the east side of the structure, in close proximity to the existing watermain. The piles should therefore be designed and socketed into shale in accordance with the design previously employed.

To reduce resistance to lateral movement and provide a relatively flexible pile system for integral abutment design, the top of each pile should be installed in a pre-augered hole supported by a CSP and filled with loose sand as per MTO Structural Office Report SO-96-01.

6.2.5 Structural Culverts

Based on review of OSIM reports, archive geotechnical borehole plans, and site observations, seven concrete box or rigid-frame culverts, two precast concrete arch culverts, and one elliptical steel pipe culvert have been identified along the Regional Road 25 corridor within the project limits. The structures are summarized below in Table 6.5.

Details regarding the culvert invert levels or founding levels of open-footing culverts (if present) were not available. Further, the number of culverts that will require extension or replacement is not known.

Table 6.5: Structural Culvert Details

Structure ID (25-1157…)	Location	Culvert Type	Fill Depth on Structure (m)	Year of Construction
510-CU01	350 m south of Upper Middle Road West	CIP concrete box, 3.6m wide x 2.5m high x 42.0m long	2.0	1980
510-CU03	200 m south of Upper Middle Road West	CIP concrete box, 2.3m wide x 1.5m high x 49.5m long	3.0	1970
560-CU-01	100 m north of Upper Middle Road West (14 Mile Creek)	Twin CIP concrete, open footing, 4.2m wide x 48.7m long	3.0	1972
640-CU01	130 m south of 407 WB Off Ramp Signal	Elliptical Steel Pipe, 2.6m wide x 1.8m high x 48m long	1.7	2008
750-CU05	400 m north of Burnhamthorpe Road	Twin precast concrete box, 2.4m wide x 1.2m high x 34m long	0.6	2013
850-CU05	200 m north of Lower Base Line Road	Precast concrete arch, 7.3m span x 1.83m high x 48.3m long	1.2	2015
860-CU05	460 m north of Halton Landfill entrance	Precast concrete arch, 6.1m span x 1.52m high x 43.01m long	2.0	2015
910-CU03	500 m north of Britannia Road	CIP concrete box, 1.8m wide x 33m long	1.8	2014
950-CU01	800 m north of Louis St. Laurent Avenue	Precast concrete box, 1.8m wide x 0.9m high x 26m long	1.0	2014
950-CU03	300 m south of Derry Road West	CIP concrete rigid-frame, 3.0m wide x 2.5m high x 45m long	3.5	1950

Based on the culvert heights and overlying fill depths indicated on the OSIM reports, and the ground elevations determined at the borehole locations, the approximate invert levels of the culverts were estimated to be as follows:

Table 6.6: Estimated Culvert Invert Elevations

Structure ID (25-1157…)	Culvert Type	Estimated Invert Level Depth (m)	Estimated Invert Level Elevation (m)
510-CU01	CIP concrete box	4.5	124.8
510-CU03	CIP concrete box	4.5	125.6
560-CU-01	Twin CIP concrete	6.3	125.9
640-CU01	Elliptical steel pipe	3.5	162.9
750-CU05	Twin precast concrete box	1.8	172.8
850-CU05	Precast concrete arch	3.0	174.7
860-CU05	Precast concrete arch	3.5	176.0
910-CU03	CIP concrete box	3.6	185.4
950-CU01	Precast concrete box	1.9	183.9
950-CU03	CIP concrete rigid-frame	6.0	184.3

**Borehole numbers from previous investigations are prefixed by the year of drilling (i.e., 07-03 indicates Borehole 3 drilled in 2007)*

In general, the subsurface stratigraphy encountered in the boreholes at the culvert locations consisted of a pavement structure and roadway embankment fill underlain by probable alluvial deposits, underlain by stiff to hard silty clay till.

For extension or replacement of box and pipe culverts, preparation of the culvert subgrade prior to placement of bedding material should include subexcavation of the fill and organic/alluvial deposits. Footings for open-footing or arch culvert extension/replacement should generally be founded on native undisturbed till below the level of the fill and organic/ alluvial materials, generally at the same level as the existing culvert footings.

The highest founding elevations and factored geotechnical resistances recommended for preliminary design of new structures or culvert extensions, based on the borehole data, are presented in Table 6.7. The design of the existing foundations should be confirmed during detailed design.

Table 6.7: Geotechnical Resistances for Culvert Foundation Design

Structure ID (25-1157…)	Reference Boreholes*	Highest Recommended Founding Level	Founding Soil	Factored Geotechnical Resistance (kPa) ULS	Factored Geotechnical Resistance (kPa) SLS
510-CU01	C-01	124.4 123.7	Stiff clay till Very dense silt till	225 450	150 300
510-CU03	C-02	126.0	Hard clay till	450	300
560-CU01	P-05, 03-5	Undetermined	--	--	--
640-CU01	C-03	162.3	Hard clay till	450	300
750-CU05	C-04	172.0	Very stiff clay till	375	250
850-CU05	C-05	173.5	Hard clay till	450	300
860-CU05	C-06	173.3	Very stiff clay till	375	250
910-CU03	07-19, 07-20	184.8	Hard clay till	450	300
950-CU01	07-09, 07-22	180.5	Dense silt till or gravelly sand	450	300
950-CU03	07-13, 07-14, 07-15	183.2	Hard silt/clay till	450	300

**Borehole numbers from previous investigations are prefixed by the year of drilling (i.e., 07-03 indicates Borehole 3 drilled in 2007)*

The subsurface conditions encountered in the reference boreholes at the Fourteen Mile Creek crossing (Structure 560-CU01) varied substantially. Previous Borehole 03-5 encountered hard clay till at 3.1 m depth however no elevations are shown on the borehole log. Current Borehole P-05 was terminated in firm alluvial material above the till surface at 8.2 m depth. If extension of this culvert is required, detailed investigation will be required to delineate the varying depth to competent native soils.

Where founding levels of adjacent footings differ, the founding elevation between footings should be stepped in maximum 0.6 m steps at a maximum inclination of 10 horizontal to 7 vertical.

A minimum 300 mm thick layer of Granular A bedding material should be provided under the base of the new culverts and extensions. The bedding thickness may need to be increased if subexcavation is required to remove fill, organic deposits and/or excessively loose/soft materials below the design base level of the culvert.

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

Backfill at the culverts should consist of non-frost susceptible, free-draining granular material conforming to OPS Granular A or Granular B Type II specifications. Reference should be made to the backfill arrangements stipulated in the OPSD 802 and 803 series as applicable.

6.3 Frost Cover

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

6.4 Lateral Earth Pressures

The backfill to abutment walls and retaining walls should be placed in accordance with OPSS.PROV 902 or OPSS.MUNI 902 as applicable, and to the extents shown in OPSD 3101.150 or 3121.150. The backfill should consist of Granular A or Granular B Type II material meeting the requirements of OPSS.PROV 1010.

The lateral earth pressures acting on the walls, assuming full drainage, 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 6.8 lists unfactored parameters for design purposes, assuming an essentially level ground surface behind and in front of the walls.

Table 6.8: Lateral Earth Pressure Parameters

Retained Material	Unit Weight (kN/m^3)	Friction Angle (degrees)	Earth Pressure Coefficient Active (Ka)	Earth Pressure Coefficient At-rest (K_o)	Earth Pressure Coefficient 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 walls are 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.

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

6.5 Detailed Geotechnical Investigation

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 existing roadway pavement and widening areas to further define the subgrade conditions, determine topsoil thickness, and confirm the pavement design recommendations. Coring of the existing asphalt and non-destructive testing such as Falling Weight Deflectometer (FWD) testing should be considered to further assess suitable rehabilitation options for specific sections of the existing pavement structure;
- Boreholes within the envelope of structure foundation units where required to confirm the subsurface conditions at the structure locations and develop detailed geotechnical recommendations for design and construction of the culvert and bridge foundations;
- Boreholes along fill embankments that require widening, potentially including the Fourteen Mile Creek crossing, Highway 407 approaches, and Sixteen Mile Creek embankment south of Derry Road, to develop recommendations regarding embankment slope stability, settlement and construction.
- Further assessment of dewatering requirements and the need for a PTTW; and
- Chemical testing of on-site soils to confirm the requirements for reuse or disposal of excavated material.

THURBER ENGINEERING LTD.

7. SIGNATURES/CLOSURE

We trust this report provides the information you require at this time. If you have any questions, please contact the undersigned at your convenience.

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THURBER

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FIGURES

APPENDIX A

Previous Geotechnical Investigations

APPENDIX B

Borehole Location Plan

APPENDIX C

Borehole Logs

APPENDIX D

Geotechnical Laboratory Results